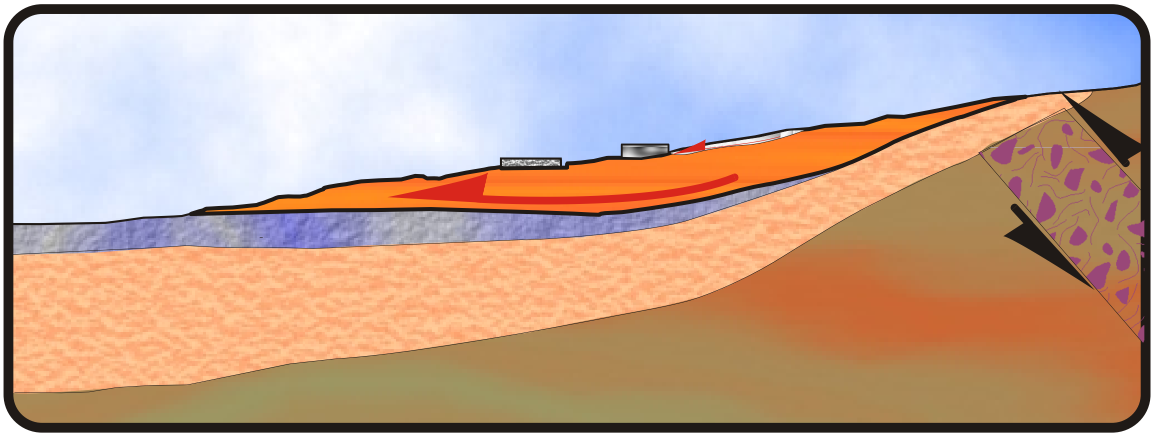
PENITENCIA CREEK LANDSLIDE
1972 through 2000 SURVEILLANCE REPORT



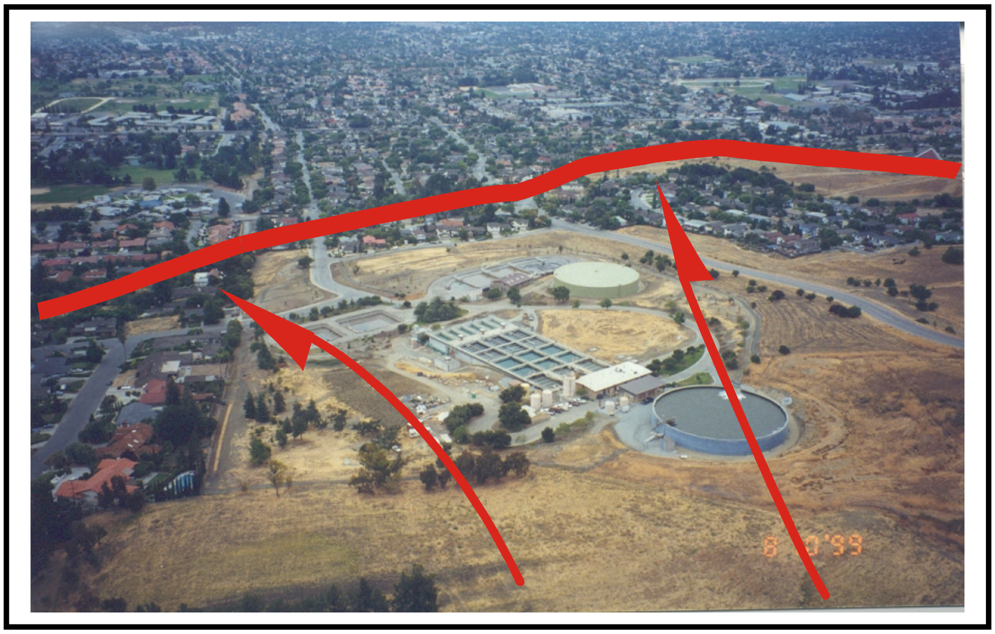
Santa Clara Valley Water District

July 2001

PENITENCIA CREEK LANDSLIDE
1972 through 2000 SURVEILLANCE REPORT

Prepared by:

Santa Clara Valley Water District
Water Utility Engineering Unit
5750 Almaden Expressway
San Jose, CA 95118



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July 2001

PENITENCIA CREEK LANDSLIDE
1972 through 2000 SURVEILLANCE REPORT

Surveillance Reporting Period:

Twelve Pneumatic Piezometers and Rainfall Data from 1972 through 2000
Electronic Distance Measurement (EDM) by DWR from 1972 through 1997
Eighteen Inclinometer Probe Data from 1985 through 2000
Penvault Pipeline Coupling Closure Data from 1986 through 2000
Four Plant Monument Survey Data from 1991 through 2000
Six Monument Line Survey Data from 1996 through 2000
Four Time Domain Reflectometry Cable Data from 2000 through 2001

The analysis and interpretation of the instrumentation and other relevant data presented in this report has involved the application of certain graphical and statistical interpretive techniques common to the geosciences and geotechnical engineering professions. The conclusions and recommendations presented in this report were developed upon evaluating the performance of all instruments and other pertinent data during the specified period, and represent the professional judgment of its authors. The draft of this report was submitted on January 25, 2001 and the final was submitted on July 31, 2001.

Report prepared by:

Originals signed by

James L. Nelson, C.E.G.
Associate Engineering Geologist
Water Utility Engineering Unit

Originals signed by

Richard L. Volpe, P.E., G.E.
Senior Engineer

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1.0 INTRODUCTION

1.1 Report Objectives

The objective of this report is to present an up-to-date understanding of the geologic and geotechnical aspects of the Penitencia Creek Landslide. The landslide is significant because it underlies the entire region, including the District's Penitencia Water Treatment Plant (PWTP, facility, or plant). It is hoped that this report will reach a varied audience; ranging from District design engineers and consultants, who are currently or will in the future be involved with upgrading the Penitencia Water Treatment Plant; to other city, county, or state agencies that may have an interest in the landslide. District staff or their consultants should use this report as a reference to assess and monitor potentially adverse impacts imposed by the landslide on the treatment plant facilities. In this manner, the Board of Directors and their constituents will have some assurance that the District is dedicated to providing a safe and reliable source of treated drinking water from the PWTP facility.

Several factors have provided impetus for publication of this report. Perhaps the most important was the need to provide a written report documenting all surveillance data and findings that have been collected at the site since 1972. Additionally, our ability to now perform and collect state-of-the-art GPS location surveys has allowed for the first time the highest degree of understanding of how various parts of the landslide are responding to ongoing movement. Recently, a question has been raised regarding the potential for unexpected shallow landslide movements during construction of the treatment plant upgrades as well as the potential for large movements during a major earthquake on nearby active faults. This report addresses these issues.

1.2 Background

The PWTP is a 42 million-gallon per day (MGD) facility that was constructed by the Santa Clara Valley Water District (SCVWD or District) in 1973/74. The facility is located on the eastern side of Santa Clara Valley about 2 miles east of Highway 680 and about 5 miles northeast of downtown San Jose, California (see Figure 1). The PWTP treats imported water that is distributed as potable water for use by much of the eastern side of the city of San Jose and surrounding areas, including the city of Milpitas.

The PWTP is situated on an approximately 240-acre creeping landslide known as the Penitencia Creek Landslide (see Figure 2). In addition to the PWTP facility, there are two other water distribution facilities located on the landslide. They include: 1) California Department of Water Resources' (DWR) 3 million gallon Terminal Reservoir for the South Bay Aqueduct built in 1965: and 2) San Jose Water Company's 5 million gallon Dutard Reservoir, a covered treated water reservoir built in 1958. Private development on the landslide includes about 250 single-family residences located on the lower one-third to one-half of the landslide and associated infrastructure such as roads, storm drains, sanitary sewer lines, and underground gas, electrical, and water lines. Landslide damage to improvements tends to have been concentrated near the toe of the landslide and has included failure of buried pipes and distress to some house foundations. Evidence of creep movement can be observed in distressed pavements, concrete curbs, buried utility lines, and houses. No specific landslide-related damage to the treatment plant structures has occurred during the 27-year period since the plant began operation, except for the damage to the large diameter pipes carrying water to and from the PWTP and the Terminal Reservoir near the toe of the landslide.

In 1972, prior to construction of the plant, the District implemented a surveillance program to monitor the magnitude, depth, and rate of potential ground movement and the depth to groundwater in and around the future plant facilities with the installation of six incinometers and twelve piezometers. The preliminary selection of instrument locations was based on knowledge of ground instability and landslide activity in the area of construction for the future water treatment plant. Although there is no record of a specific report that dealt with the landslide, the District went forward with the construction knowing that the landslide was active but apparently moving at a very slow velocity. The District recognized the need for a long-term landslide-monitoring program in 1975, when the initial inclinometers that had been installed in 1972 were sheared and could no longer be read. Since 1972, the District has completed 28 inclinometer borings at locations shown in Figure 3 and 26 pneumatic piezometers at the locations shown in Figure 4. In addition, surface ground movement has been monitored by ground surveys (electronic distance measurement or EDM) at selected points by the DWR since 1972. The locations of the EDM monitoring points are shown in Figure 5.

In 1983, the District began monitoring the convergence of several pipe compression joints near the toe of the landslide after the pipes had failed in May 1983 by landslide-induced compression of the pipes. The pipe compression monitoring points are located in the concrete “Penvault”, a buried concrete structure needed for maintenance, at the location shown in Figure 5. Locations of all inclinometers and piezometers are shown in cross section in Figures 6 through 10. Since 1991, District surveyors have used GPS to survey locations of four monuments situated near the four corners of the water treatment plant (Figure 5). These monuments have been destroyed during the current upgrade work at the plant. New monuments will be established on the upgraded plant. Additionally, since 1996, surveyors have used GPS to survey monuments along six survey lines established at various locations on the landslide. Each line consists of multiple “shiner and nail” monuments positioned along roughly linear street alignments at the locations shown in Figure 5. The data from these monuments has been very helpful in locating the toe of the landslide.

During the extended period of time that the surveillance techniques discussed above were implemented, various technical analyses have been performed, and memoranda and reports have been written by District personnel, or by District consultants, to aide in the interpretation of landslide movement or characteristics. In preparation of this report, we attempted to collect and review as much historical data and references as could be located in the District library or files. Appendix A presents a detailed annotated reference list and chronology of the technical investigations and related work for both the PWTP and the Penitencia Creek Landslide. This report represents the first written surveillance report prepared by the District to describe the monitoring results for all of the various instruments installed at the landslide, and other District related efforts to monitor the landslide, since first beginning the monitoring effort in 1972. In addition to the surveillance data, the report also presents a detailed discussion of the geologic and geotechnical conditions that most likely lead to the development and propagation of the landslide.

1.3 Current and Future District Projects

This report has been prepared to provide specific geotechnical and geological information to support current and future projects such as those associated with the current Treated Water Improvement Project (TWIP). The PWTP, along with the District's other two water treatment facilities, is currently undergoing a major upgrade as part of the TWIP project. The work has been divided into two broad phases for design and stages for construction. The Stage 1 TWIP work, which is currently under construction and is estimated to be finished by June 2002, will provide process improvements for water treatment and includes structural upgrades of the plant process basins to address landslide-and-earthquake-loading related issues. Phase 2 TWIP design work will add ozone disinfection and other new facilities to further improve water quality and improve plant reliability with piping modifications to address landslide-and- earthquake-loading related issues. Stage 2 TWIP construction is scheduled for completion by November 2004. It should be noted that many of the results and conclusions that are formalized in this report were already known and properly incorporated into the final design of the Phase 1 and Phase 2 TWIP work by District geotechnical consultants (Woodward-Clyde Consultants in 1993 for Phase 1; and Harza Engineering Company in 2000 for Phase 2). Some of the newer findings of this report, especially the location and orientation of the toe-of-slide movement, are currently being addressed for future Stage 2 TWIP designs.

1.4 History of Land Use

The Penitencia Creek Landslide and nearby areas were used for orchards and grazing from the early twentieth century through the early 1950s. Suburban growth reached the landslide vicinity in the 1950s as the City of San Jose grew in geographic size and population, expanding from the relatively easy to develop Santa Clara Valley floor onto the adjacent hill slopes. Stereoscopic aerial photographs of the project area are available for scattered dates, starting in 1939. Based on an analysis of these photographs, the land use was orchards and grazing through about 1960. The 1960 photographs show about 30 houses under construction at various locations on the landslide. Photographs show that the present-day level of housing development (about 250 homes) was fully attained by about 1971. A listing of the aerial photographs reviewed in preparation of this report is presented in Appendix D.

As a wholesale water supply and water resources management agency, the District receives a significant portion of its imported water from the State Water Project. Water from the State Water Project is delivered to the District via the South Bay Aqueduct (SBA) to the Terminal Reservoir adjacent to the PWTP. The SBA was completed in 1964 when the Terminal Reservoir, a 401,000 ft^3 (3.0 million gallons) open steel tank, was built on the future site of the Penitencia Water Treatment Plant. Early siting studies for the Terminal reservoir and the future PWTP evaluated several sites in the region (Santa Clara County Flood Control and Water Conservation District 1962, Marliave 1963, and Dames & Moore, 1972). Generally, these authors did not recognize or acknowledge the existence of the large, deep, landslide that came to be known as the Penitencia Creek Landslide.

After the construction of structures that were sensitive to surface land movements, such as house foundations and concrete sidewalks and street curbs, the existence and creep activity of the Penitencia Creek Landslide became apparent when cracks and displacements appeared by the early 1970s. Detailed field mapping and photo-interpretation by Nilsen & Brabb (1972) provided the first authoritative published map that clearly delineated the existence of the Penitencia Creek Landslide and several other major landslides in the area.

State and federal requirements for Environmental Impact Reports resulted in numerous investigations and evaluations of the Penitencia Creek Landslide beginning in about 1969. Clearly, the existence of landslide hazards, and the need to control land development to minimize damage to public and private property, became apparent by the permitting authorities in the mid-1970's. In 1976, the City of San Jose adopted an ordinance requiring geologic reports for hillside development as part of the permitting process (Nelson, 1976). The city adopted a Geologic Hazard Clearance process in 1994, and with modifications in 1997. While every investigation or report can not be summarized herein, we note that some consultants and geologists in their reports have not readily accepted the existence of the Penitencia Creek Landslide. The controversy of whether or not Penitencia Creek Landslide existed was brought closer to closure with three major studies: Woodward-Clyde Consultants (1993), City of San Jose Geologic Review Panel (1994), and Norfleet Consultants et al. (1995). These studies essentially accepted or confirmed the existence of the landslide as generally envisioned by earlier authors such as Nilsen & Brabb (1972), Wahler Associates (1982), and several others.

1.5 Report Organization

This report consists of nine chapters and four appendices. Following this introductory chapter, Chapter 2 presents a summary of the site conditions and hydrology. Chapter 3 presents a discussion of regional and site geology and regional seismicity. A discussion of the geotechnical engineering properties of the slide materials is presented in Chapter 4. Chapter 5 presents a discussion the various types of instruments and survey methods used to determine the magnitude and rate of landslide movement at the site. Chapter 6 presents a discussion of the analyses and results during the 28+ years of monitoring efforts at the landslide. Chapter 7 presents a discussion of the landslide geometry and the most likely landslide mechanism. Chapter 8 acts as an Executive Summary and presents conclusions and recommendations based on results presented elsewhere in the report. Chapter 9 presents an alphabetical list of references cited in the text or appendices. Appendix A presents an annotated reference list and chronology of work performed at the landslide. Appendix B presents detailed piezometer results. Appendix C presents detailed inclinometer results. Appendix D presents a table of 28 aerial photographs reviewed for the project. Several historic aerial photographs are also presented in Appendix D that have been augmented to show the limits of the slide and other specific features.

The text is paginated in accordance with the chapter and sequential page number (e.g., the fifth page in chapter one is 1-5, etc.). Throughout the text, tables and figures that could be contained on one page are interspersed throughout the text. Although the tables are numbered, and referenced in the table of contents, the smaller schematic figures are not. Tables and figures are sequentially numbered. The text references a total of 51 figures. They are placed sequentially following the last page of the chapter in which they are first referenced. The appendices also contain numerous tables and figures. A description of what is sequentially presented in each appendix is presented in the table of contents.

1.6 Vertical Datum

Two vertical datum's are used in this report. The older datum is the National Geodetic Vertical Datum (NGVD) 1929, also referred to as Mean Sea Level or the National Coast and Geodetic Datum. The newer datum is the North American Vertical Datum (NAVD) 1988. For the PWTP, the relationship between the two datum's is NAVD (1988)= NGVD (1929)+ 2.75 feet. The specific datum used for each figure is referenced, where appropriate.

1.7 Acknowledgments

Tom Iwamura and Robert E. Tepel, both retired engineering geologists from the District, completed the initial instrumentation and many of the early studies of the landslide. Messrs. Iwamura and Tepel advanced a clear understanding of the nature and magnitude of movement and their major efforts are acknowledged. Gary Faler (retired), Steve Choy, Dan Gilleland and Kris Puthoff, all of the District's Survey Department, have provided valuable assistance with both survey and inclinometer data. Brian O'Mara, Don Naccarato and Robert Ramento, from the Operations Planning and Analysis Unit, have provided valuable assistance with the piezometer data. Many other District personnel have participated through the years and their major efforts in collecting instrumentation results are acknowledged.

We wish to acknowledge the engineering assistance of John Mueller, P.E. (retired) who reviewed and summarized the historical data sources and also helped in preparing Appendices A and B.

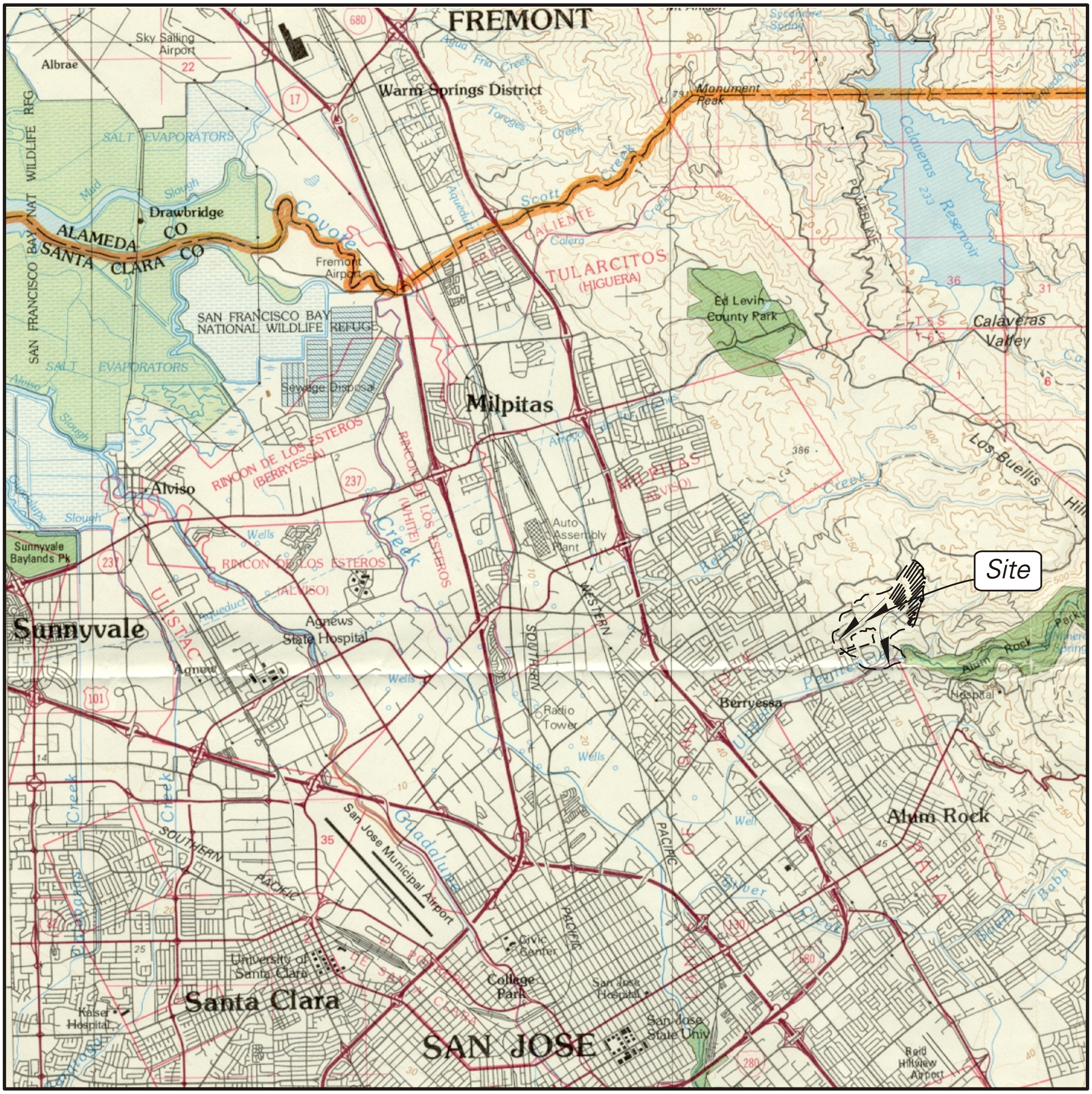
We wish to acknowledge the collaborative efforts of three local engineering geologist experts: David Andersen, San Jose State University; and Carl Wentworth and John Tinsley, III with the U.S. Geological Survey. Professor Andersen is a sedimentologist with expertise in the Santa Clara Formation, Dr. Tinsley specializes in Quaternary geology and fluvial processes, and Dr. Wentworth is a regional geologist with significant expertise in local Bay Area geology. These three prominent engineering geologists provided useful information to the authors dealing with discussion of possible geologic history and factors that contributed to the development of the Penitencia Creek Landslide presented in Chapter 3.

Finally, we wish to acknowledge the principal reviewers, both District and external, who provided valuable comments to improve the quality of the report. Especially, we wish to acknowledge the considerable efforts of the Department of Resources, including Rob Berry, Senior Engineering Geologist and Don Walker, Associate Civil Engineer for their very thorough technical review.

1.8 Limitations

This report has been prepared by the Santa Clara Valley Water District and is intended for the exclusive use of District staff and consultants. Others should make no reference to any of the data or findings of this report without prior written authorization from the District.

This report describes landslide movements and other geotechnical and geological issues that are occurring at the Penitencia Creek Landslide and that are affecting the District's Penitencia Water Treatment Plant. The District and others, in order to monitor various aspects of the landslide, have installed numerous geotechnical instruments and survey monuments throughout the landslide over time. By publishing this report, the District wishes to emphasize that it is only interested and concerned about movement of the Penitencia Creek Landslide that is physically occurring within District property limits and its potential affects upon District facilities. The District is not responsible for other possible interpretations of the technical or interpretive data, findings or conclusions presented in this report that could affect adjacent or nearby properties.



Base map: U.S.G.S. 1:100,000-Scale Series (topographic), San Jose California Quadrangle, 1968-1973, revised 1978

File: W:\geology\penitencia creek landslide\figures\fig1.cdr

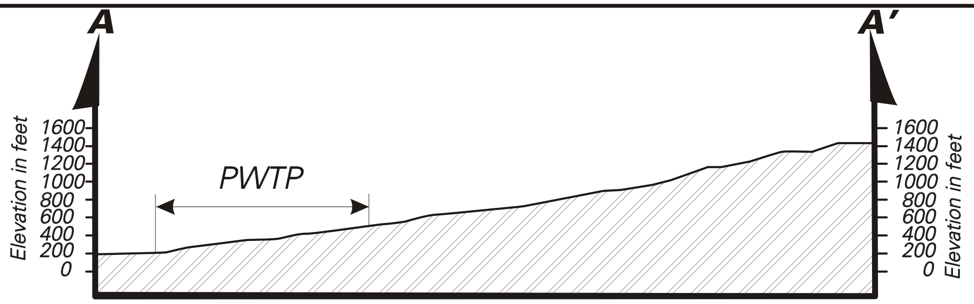
DATE 1/8/01	Santa Clara Valley Water District	
DESIGN JLN		
DRAWN JLN		
CHECKED RLV		

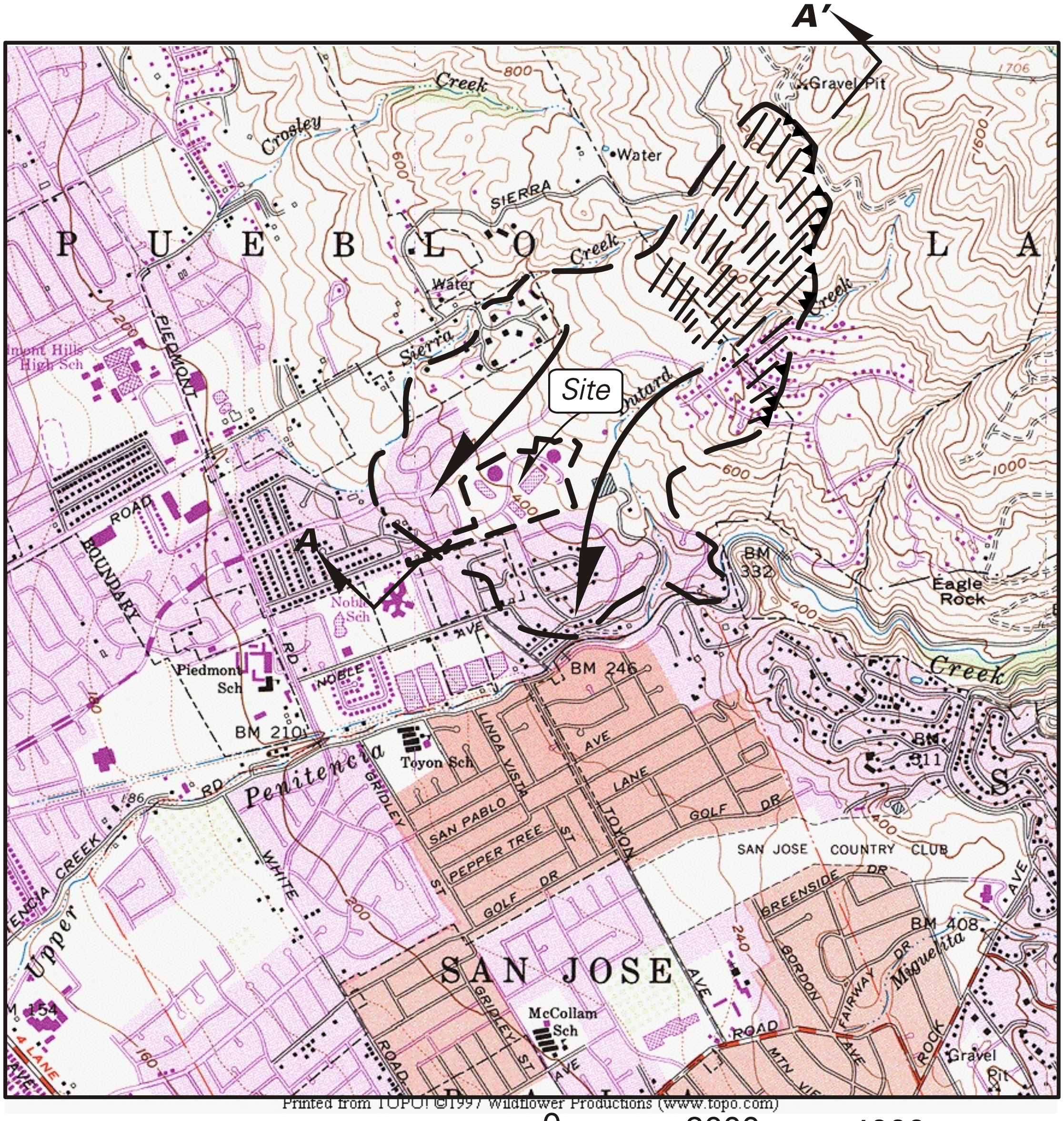
Regional Map

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=1.61 mi

FIGURE
1





0 2000 4000

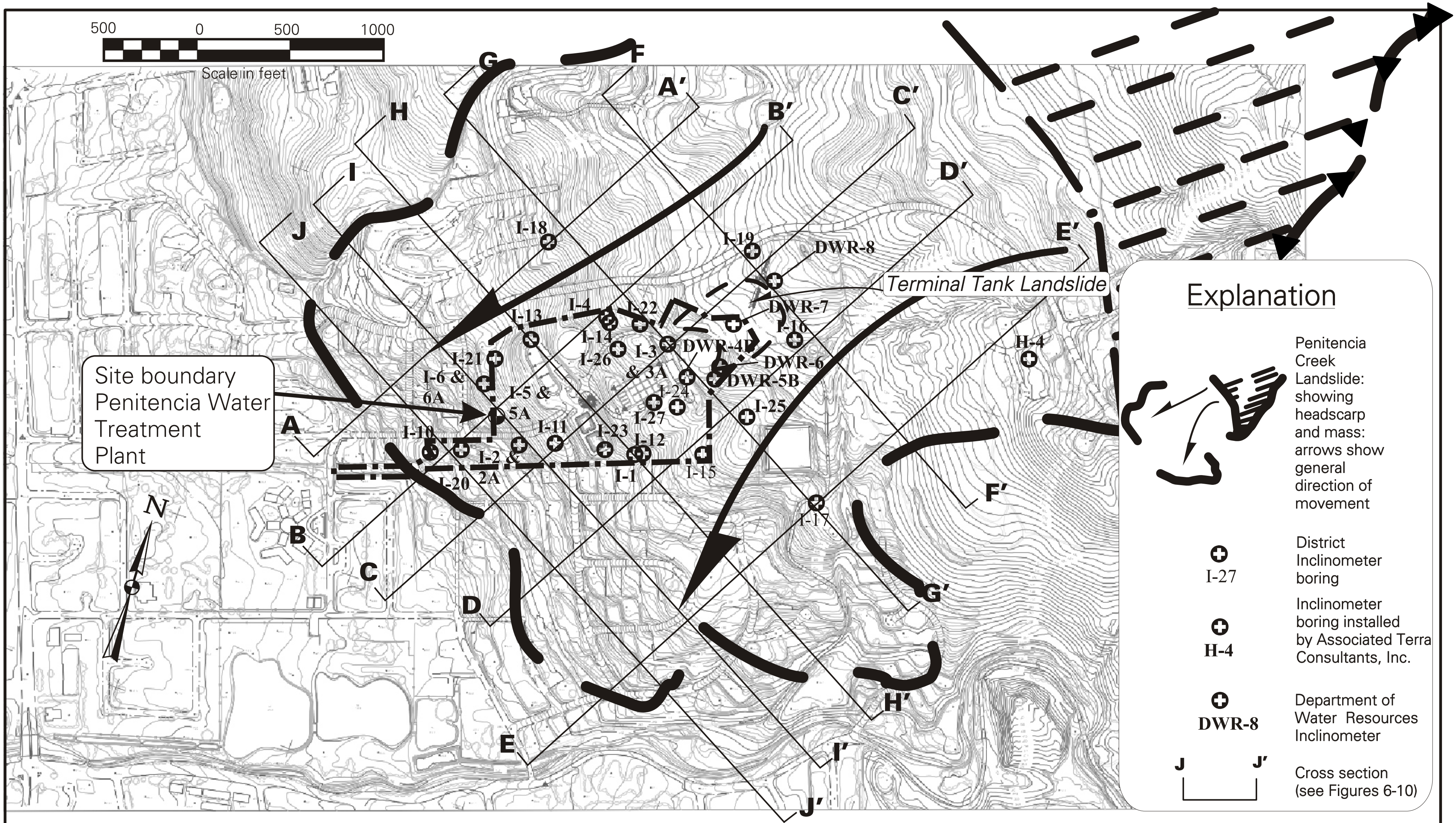
Scale in Feet

N

Base map: U.S.G.S. 7.5 minute Calaveras Reservoir,
California quadrangle, 1961, revised 1980

File: W:\geology\penitencia creek landslide\figures\fig2.cdr

DATE 11/8/00	Santa Clara Valley Water District	Site Map Penitencia Creek Landslide Penitencia Water Treatment Plant	SCALE 1"=2000'
DESIGN JLN			FIGURE 2
DRAWN JLN			
CHECKED RLV			



500
0
500
1000
Scale in feet
N
G
F
A′
B′
C′
D′
E′
F′
G′
H′
I′
J′
H
I
J
A
B
C
D
E
Site boundary Penitencia Water Treatment Plant
Terminal Tank Landslide
I-18
I-19
DWR-8
DWR-7
I-16
DWR-6
DWR-5B
DWR-4B
I-4
I-22
I-13
I-14
I-26
I-3 & 3A
I-21
I-6 & 6A
I-5 & 5A
I-24
I-27
I-25
I-10
I-2 & 2A
I-11
I-23
I-12
I-20
I-1
I-15
I-17
H-4
Explanation
Penitencia Creek Landslide: showing headscarp and mass: arrows show general direction of movement
I-27
District Inclinometer boring
H-4
Inclinometer boring installed by Associated Terra Consultants, Inc.
DWR-8
Department of Water Resources Inclinometer
J
J′
Cross section (see Figures 6-10)

Notes: 1) Base map by Towill, Inc., elevations based on 1929 NGVD datum,
Date of photography 1/28/84, California Coordinate System Zone 3
2) File: W:\geology\penitencia creek landslide\figures\fig3.cdr

DATE	9/28/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

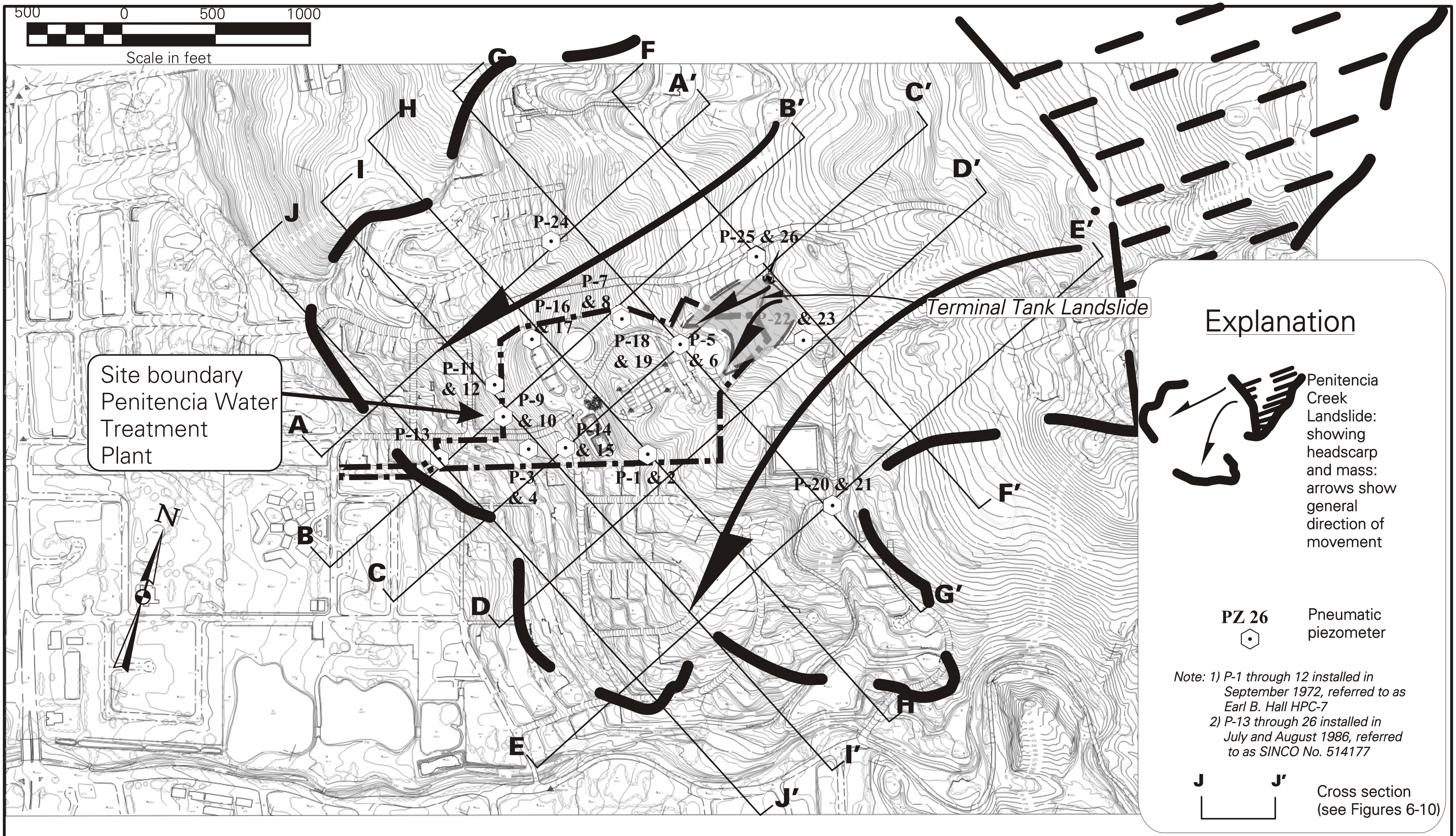
Inclinometer Boring Locations

Penitentia Creek Landslide
and Vicinity

Penitencia Water Treatment Plant

SCALE
1"= 500′

FIGURE
3



Note: 1) P-1 through 12 installed in September 1972, referred to as Earl B. Hall HPC-7
2) P-13 through 26 installed in July and August 1986, referred to as SINCO No. 514177

Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum, Photography Air Flight Service, flown 12/99, California Coordinate System Zone 3
2) File: W:\geology\penitencia creek landslide\figures\fig4.cdr

DATE 9/28/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

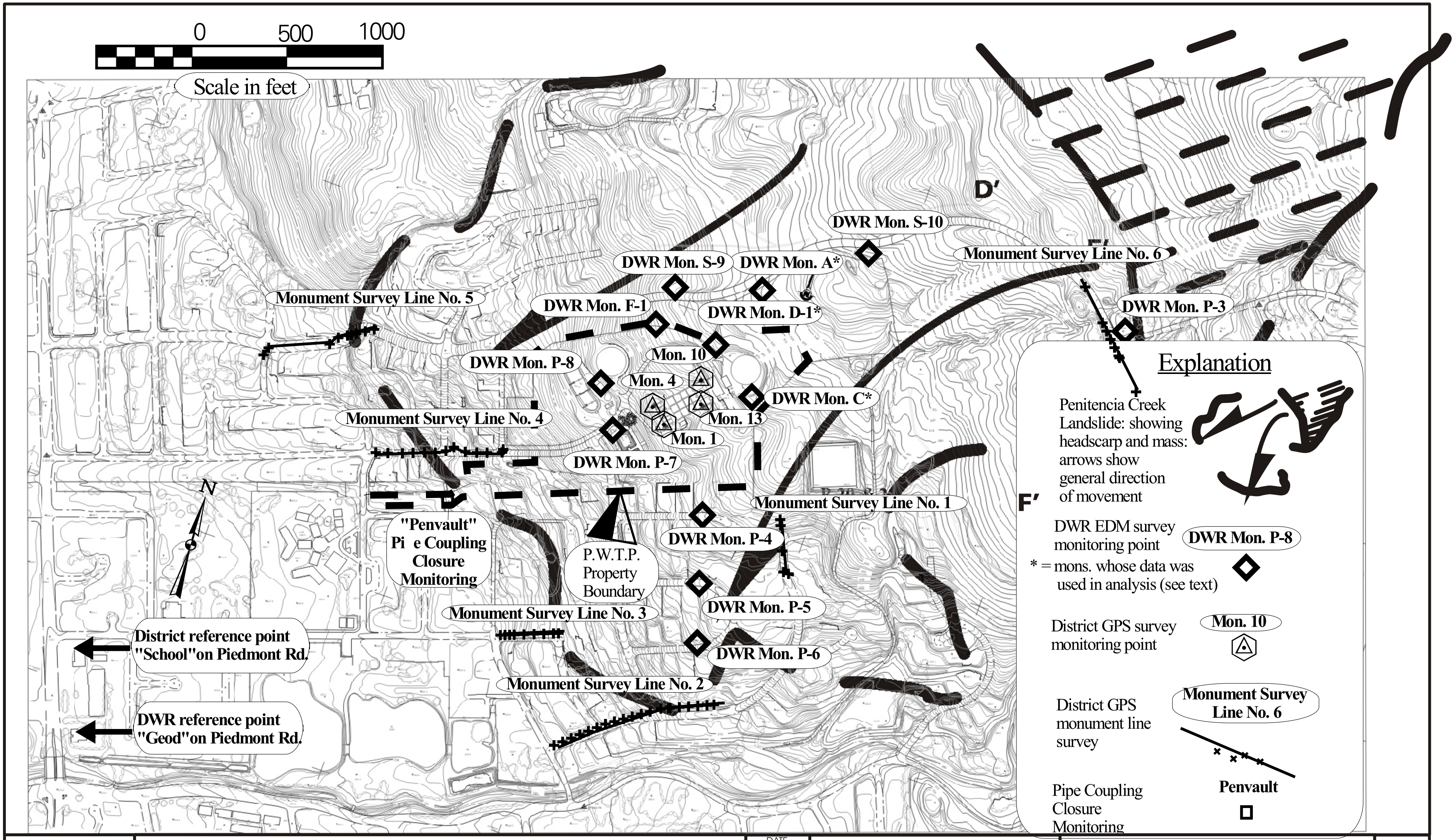
Santa Clara Valley Water District

Piezometer Locations

Penitentia Creek Landslide
and Vicinity
Penitencia Water Treatment Plant

SCALE 1"= 500′

FIGURE 4


0
500
1000
Scale in feet
D′
F′
F′
Monument Survey Line No. 6
DWR Mon. S-10
DWR Mon. S-9
DWR Mon. A*
Monument Survey Line No. 5
DWR Mon. F-1
DWR Mon. D-1*
DWR Mon. P-3
DWR Mon. P-8
Mon. 10
Mon. 4
DWR Mon. C*
Monument Survey Line No. 4
Mon. 13
Mon. 1
DWR Mon. P-7
Monument Survey Line No. 1
N
"Penvault"
Pipe Coupling
Closure
Monitoring
P.W.T.P.
Property
Boundary
DWR Mon. P-4
DWR Mon. P-5
Monument Survey Line No. 3
District reference point
"School"on Piedmont Rd.
DWR Mon. P-6
Monument Survey Line No. 2
DWR reference point
"Geod"on Piedmont Rd.
Explanation
Penitencia Creek Landslide: showing headscarp and mass: arrows show general direction of movement
DWR EDM survey monitoring point
DWR Mon. P-8
* = mons. whose data was used in analysis (see text)
District GPS survey monitoring point
Mon. 10
District GPS monument line survey
Monument Survey Line No. 6
Pipe Coupling Closure Monitoring
Penvault

Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum
Photography by Air Flight Service, flown 12/99, California Coordinate System Zone 3
2) File: W:\geology\penitencia creek landslide\figures\fig5b.grf

DATE	09/27/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

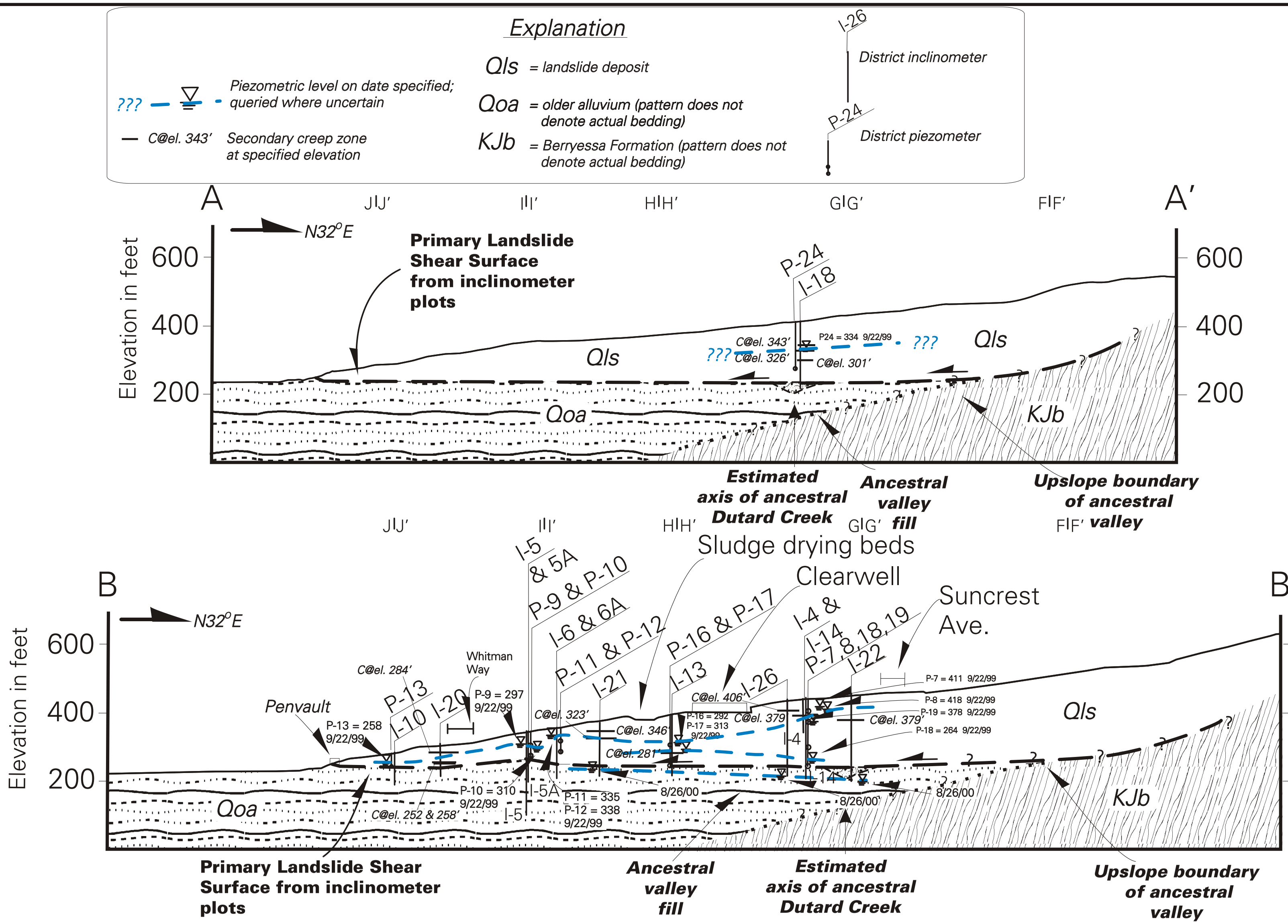
Santa Clara Valley Water District

Surface Monitoring Points Monument Lines & Pipe Coupling Closure Locations

Penitencia Creek Landslide
and Vicinity
Penitencia Water Treatment Plant

SCALE 1"=500′

FIGURE 5



Notes: 1) see Figures 3 and 4 for location of sections
2) datum based on NGVD 1929 datum which is 2.75' lower than NAVD 1988
3) file: W:\geology\penitencia creek landslide\figures\fig6.cdr

DATE	9/28/00
DESIGN	RLV
DRAWN	JLN
CHECKED	RLV

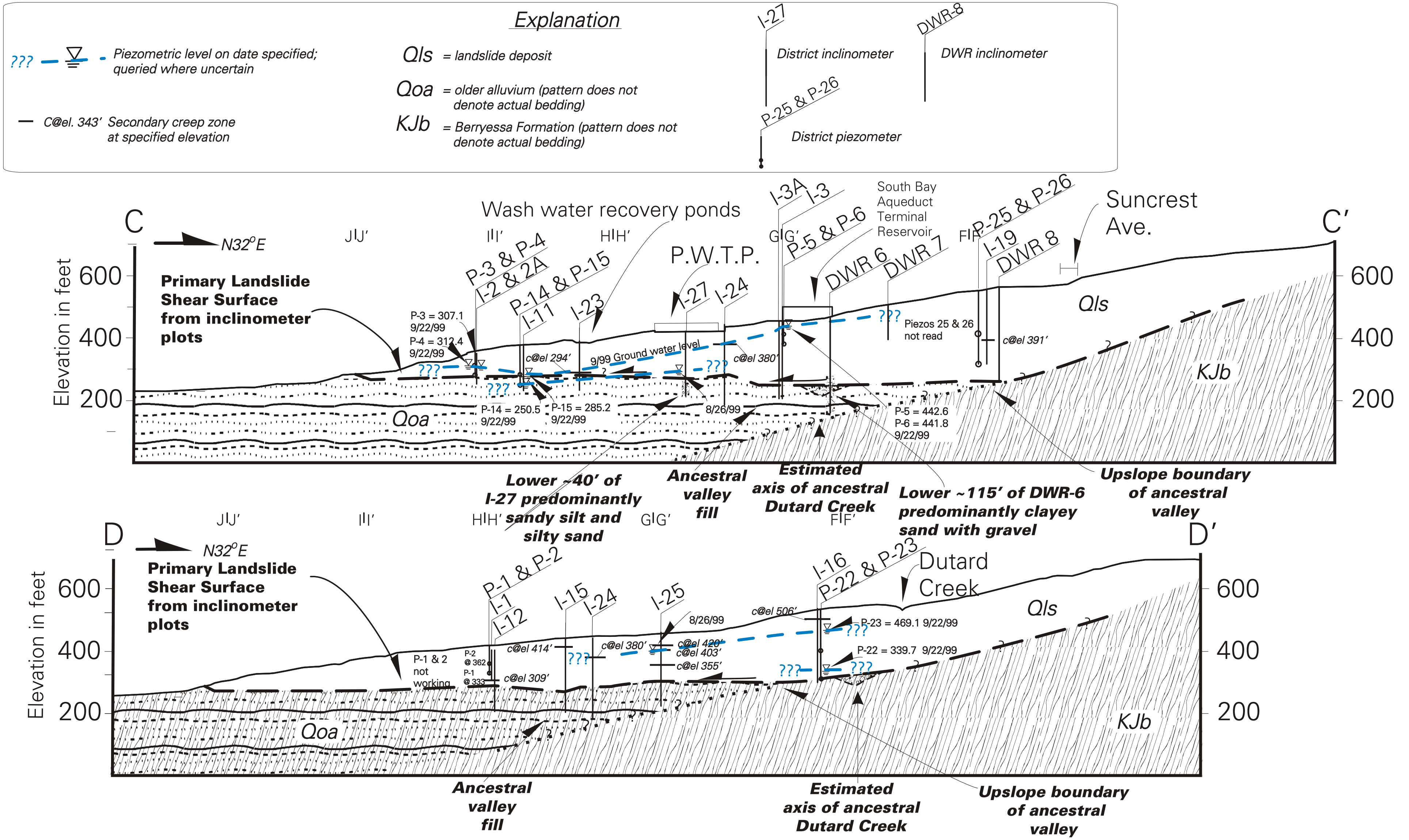
Santa Clara Valley Water District

Cross Sections A-A' and B-B'

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=300'

FIGURE
6


Explanation
??? Piezometric level on date specified; queried where uncertain
C@el. 343' Secondary creep zone at specified elevation
Qls = landslide deposit
Qoa = older alluvium (pattern does not denote actual bedding)
KJb = Berryessa Formation (pattern does not denote actual bedding)
I-27 District inclinometer
DWR-8 DWR inclinometer
P-25 & P-26 District piezometer
C
C'
N32°E
Elevation in feet
600
400
200
Primary Landslide Shear Surface from inclinometer plots
J|J'
I|I'
H|H'
G|G'
F|F'
Wash water recovery ponds
P.W.T.P.
South Bay Aqueduct Terminal Reservoir
Suncrest Ave.
P-3 & P-4
I-2 & 2A
P-14 & P-15
I-11
I-23
I-27
I-24
I-3A
I-3
P-5 & P-6
DWR 6
DWR 7
P-25 & P-26
I-19
DWR 8
P-3 = 307.1 9/22/99
P-4 = 312.4 9/22/99
c@el 294'
9/99 Ground water level
c@el 380'
???
Piezos 25 & 26 not read
c@el 391'
P-14 = 250.5 9/22/99
P-15 = 285.2 9/22/99
8/26/99
P-5 = 442.6
P-6 = 441.8
9/22/99
Qls
Qoa
KJb
Lower ~40' of I-27 predominantly sandy silt and silty sand
Ancestral valley fill
Estimated axis of ancestral Dutard Creek
Lower ~115' of DWR-6 predominantly clayey sand with gravel
Upslope boundary of ancestral valley
D
D'
P-1 & P-2
I-1
I-12
I-15
I-24
I-25
I-16
P-22 & P-23
Dutard Creek
P-1 & 2 not working
P-2 @ 362
P-1 @ 333
c@el 414'
c@el 309'
c@el 380'
8/26/99
c@el 420'
c@el 403'
c@el 355'
c@el 506'
P-23 = 469.1 9/22/99
P-22 = 339.7 9/22/99
Qoa
Qls
KJb
Ancestral valley fill
Estimated axis of ancestral Dutard Creek
Upslope boundary of ancestral valley

Notes: 1) see Figures 3 and 4 for location of sections
2) datum based on NGVD 1929 datum which is 2.75' lower than NAVD 1988
3) file: W:\geology\penitencia creek landslide\figures\fig7.cdr

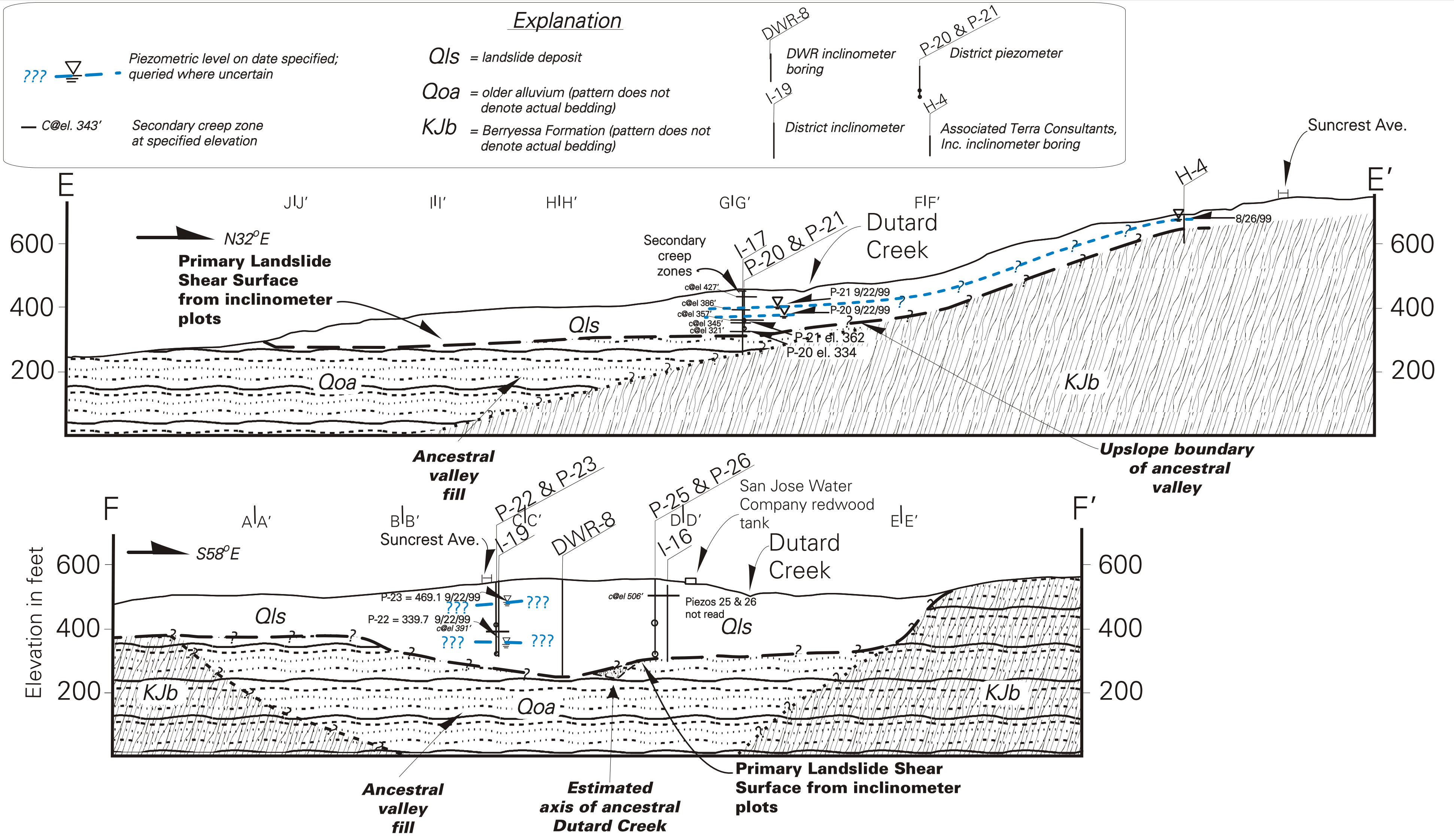
DATE 9/28/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

Cross Sections C-C' and D-D'

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=300'
FIGURE 7


Explanation
??? Piezometric level on date specified; queried where uncertain
C@el. 343' Secondary creep zone at specified elevation
Qls = landslide deposit
Qoa = older alluvium (pattern does not denote actual bedding)
KJb = Berryessa Formation (pattern does not denote actual bedding)
DWR-8 DWR inclinometer boring
I-19 District inclinometer
P-20 & P-21 District piezometer
H-4 Associated Terra Consultants, Inc. inclinometer boring
E
E'
J|J'
I|I'
H|H'
G|G'
F|F'
N32°E
Primary Landslide Shear Surface from inclinometer plots
Secondary creep zones
I-17
P-20 & P-21
Dutard Creek
c@el 427'
c@el 386'
c@el 357'
c@el 345'
c@el 321'
P-21 9/22/99
P-20 9/22/99
P-21 el. 362
P-20 el. 334
H-4
8/26/99
Suncrest Ave.
Qls
Qoa
KJb
Ancestral valley fill
Upslope boundary of ancestral valley
600
400
200
F
F'
A|A'
B|B'
Suncrest Ave.
S58°E
P-22 & P-23
C|C'
I-19
DWR-8
P-25 & P-26
D|D'
I-16
San Jose Water Company redwood tank
E|E'
Dutard Creek
P-23 = 469.1 9/22/99
P-22 = 339.7 9/22/99
c@el 391'
???
c@el 506'
Piezos 25 & 26 not read
Qls
Qoa
KJb
Elevation in feet
Ancestral valley fill
Estimated axis of ancestral Dutard Creek
Primary Landslide Shear Surface from inclinometer plots

Notes:
1) see Figures 3 and 4 for location of sections
2) datum based on NGVD 1929 datum which is 2.75' lower than NAVD 1988
3) file: W:\geology\penitencia creek landslide\figures\fig8.cdr

DATE	9/28/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

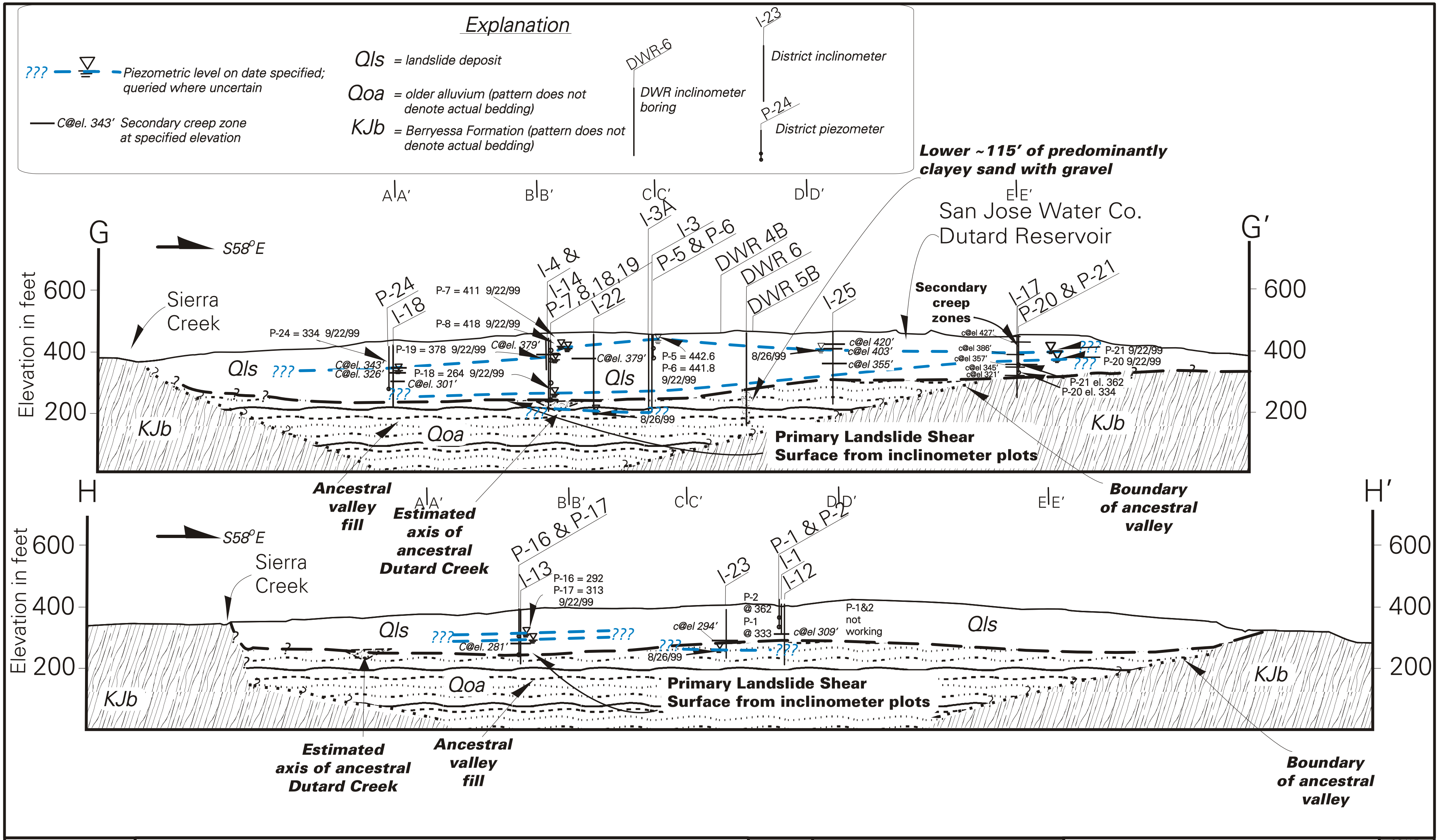
Santa Clara Valley Water District

Cross Sections E-E' and F-F'

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=300'

FIGURE 8



Notes:
1) see Figures 3 and 4 for location of sections
2) datum based on NGVD 1929 datum which is 2.75' lower than NAVD 1988
3) file: W:\geology\penitencia creek landslide\figures\fig9.cdr

DATE	9/28/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Cross Sections G-G' and H-H'

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=300'

FIGURE
9

Explanation

- ??? ▽ Piezometric level on date specified; queried where uncertain
- C@el. 343′ Secondary creep zone at specified elevation

Explanation

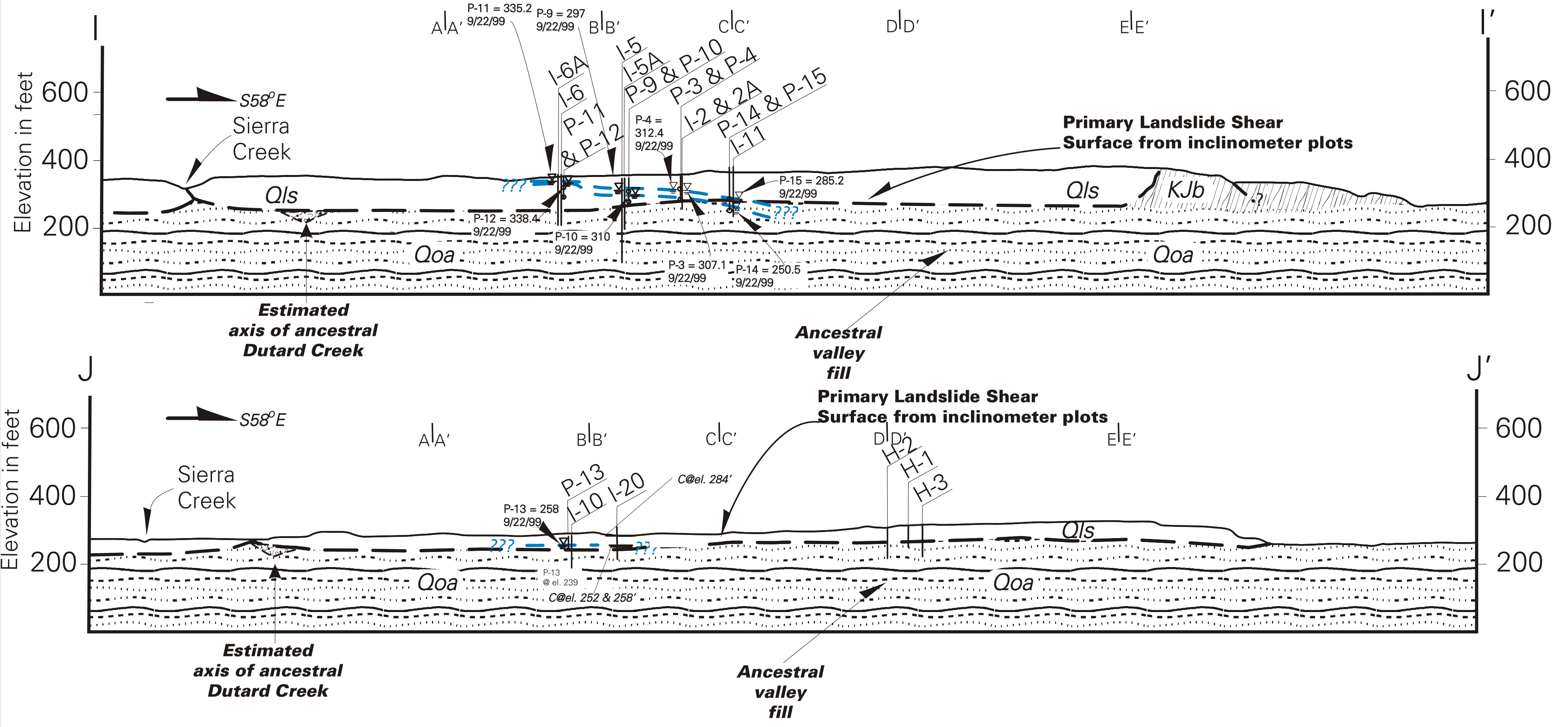
Qls = landslide deposit

Qoa = older alluvium (pattern does not denote actual bedding)

KJb = Berryessa Formation (pattern does not denote actual bedding)

I-20 District inclinometer

P-14 & P-15 District piezometer



Notes:
1) see Figures 3 and 4 for location of sections
2) datum based on NGVD 1929 datum which is 2.75′ lower than NAVD 1988
3) file: W:\geology\penitencia creek landslide\figures\fig10.cdr

DATE 9/28/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

Cross Sections I-I′ and J-J′

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=300′

FIGURE 10

2.0 SITE CONDITIONS AND HYDROLOGY

2.1 Physiographic Setting and Topography

The Penitencia Creek Landslide is located about 5 miles northeast of downtown San Jose and 1½ miles east of Interstate 680 (Fig. 1). The landslide is situated along the eastern edge of the San Francisco Bay-Santa Clara Valley depression, a poorly defined buried structure that is bounded by sub-linear fault zones and mountain ranges. The northwest-trending Santa Clara Valley is bounded to the northeast by the Diablo Range and to the southwest by the Santa Cruz Mountains.

As discussed in Chapter 1, the Penitencia Creek Landslide occupies about 240 acres of land just north of Penitencia Creek in the northeast section of San Jose. The slide is roughly rectangular in shape (slightly lobate-shaped at the downslope western edge where it contacts the valley floor) and measures about 4,400 feet (northeast-southwest) by 3,000 feet (northwest-southeast). The District's Penitencia Water Treatment Plant occupies about 24 acres of the lower (southwestern edge) of the slide. Similar other major range-front slides are located both north and south of the Penitencia Creek Landslide.

In order to describe the uniform and mild topographic variation within the slide area more easily, a cross section was developed normal to slope contours (see Figure 2). A review of Figure 2 indicates that the elevation change from the toe of the landslide to the headscarp varies from about elevation (el.) 320 feet to el. 800 feet. The ground surface through the landslide area is relatively uniform and inclined at a slope of about 7(H) to 1(V) (8.1 degrees) up to about el. 900 feet just east of the Berryessa Fault Zone. The slope inclination then steepens to about 4.5(H) to 1(V) (12.5 degrees) up to el. 1700 just east of the Hayward Fault. The crest of the Los Buellis Hills, at about el. 2000 feet, is located 1.7 miles east of the Terminal Reservoir.

2.2 Weather and Rainfall Patterns

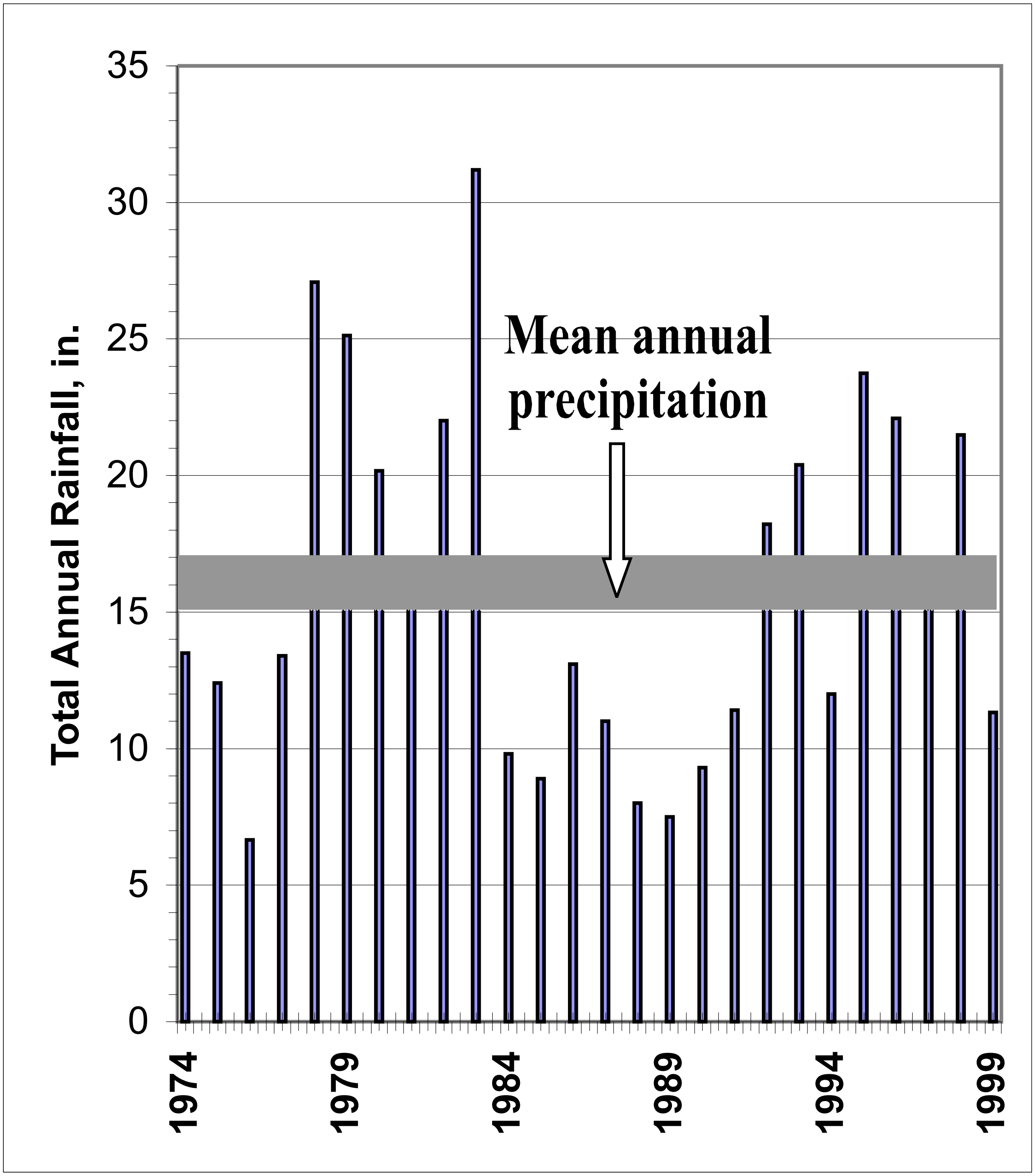
The San Francisco Bay Area's Mediterranean climate is characterized by mild wet winters and warm, dry summers. Most rainfall occurs between November and March. Rain during the summer months of June, July and August normally totals less than 0.20". Wet seasons are cool, but mild. Dry season weather is very consistent, with warm sunny days. This type of climate is common on western continental coasts between latitude 30° and 50° N. where coastal ocean currents moderate the effects of seasonal changes in temperature. The location of the anticyclonic (high pressure) cell controls the seasonal distribution of precipitation. Normally (particularly in summer), the anticyclonic cell is situated off the California coast blocking storm systems from the Gulf of Alaska. Winter precipitation occurs when the cell is absent or situated far south of its normal position. Almost all precipitation occurs as rain with rare snowfall that rapidly melts (Helley et al. 1979).

The mean annual precipitation for the Penitencia Water Treatment Plant area between 1867 and present has been between 15 and 17 inches (SCVWD, 1988). The mean annual rainfall is recognized to increase from about 15 inches in the middle of the Santa Clara Valley (San Jose) to above 31 inches at Mt. Hamilton (at elevation 4211 feet). Based on historical records (SCVWD, 1988), the annual rainfall of the upper slopes above the landslide increase to between 20 and 22 inches near the crest of Los Buellis Hills.

The District installed its first rainfall gauge at the PWTP (Station 99) in October 1967; however, the station was moved about 300 feet south and 75 feet lower in elevation in December 1974. The annual rainfall record of Station 99 from 1974 through 1999 is presented in Figure 11. The average rainfall for the 26-yr period presented in Figure 11 is 17.3 in/yr. A review of Figure 11 indicates what is considered to represent a typical variation in annual rainfall from the norm. The first four years of record (1974 through 1977) reflect below normal rainfall, ranging from 41% to 84% based on an assumed average annual rainfall of 16 inches. The next six years (1978 through 1983) reflect very wet years with annual rainfall ranging between 100% and 194% of normal. This wet period is followed by eight years of drought (1984 through 1991) where the annual rainfall ranged between 47% and 81% of normal. From 1992 through 1999, there have been five years of above average rainfall (116% to 150%) and three years of below average rainfall (69% to 94%). The impact/correlation of rainfall as a triggering mechanism for landslide movement is further discussed in Chapter 7.

2.3 Runoff Hydrology

As shown in Figure 2, the approximate lower half of the landslide has city street development that affects the collection and disposal of distributed rainfall. There are three major creeks that drain the landslide area. Sierra Creek, which flows essentially east west, is about 7,000 feet-long. It is located on the north side of the landslide and actually cuts across the northeast corner of the slide headscarp and then terminates near the Hayward fault. Dutard Creek is located about 1,800 feet to the south of Sierra Creek and roughly parallels it over a length of about 3,000 feet. It begins uphill of the slide headscarp at about el. 1100 feet and then forms the northeastern slide boundary. At about el. 520 feet, and at a point about 700 feet uphill from the DWR Terminal Reservoir, Dutard Creek is deflected to the south in the area where it encounters the Clayton fault. As will be discussed in more detail later in this report, one lobe of the landslide also bends to south at about the same location where Dutard Creek deflects to the south. Both Sierra Creek and Dutard Creek could act as primary sources of groundwater recharge to the slide. A comparison of stream flow measurements upstream and downstream of the slide is necessary to determine the extent, if any, that these creeks recharge the landslide. Dutard Creek discharges into Penitencia Creek, which forms the southern boundary over lower southwestern section of the landslide. Penitencia Creek, located southwest of the slide, is a significantly larger drainage than either Sierra Creek or Dutard Creek.



DATE 1/10/01	Santa Clara Valley Water District	*Annual Rainfall for STA 99* *from 1974 through 1999* *Penitencia Creek Landslide* *Penitencia Water Treatment Plant*	SCALE none
DESIGN JLN			FIGURE 11
DRAWN JLN			
CHECKED RLV			

3.0 GEOLOGY AND SEISMICITY

3.1 Regional Geology

The San Francisco Bay Area is situated at the tectonic junction of the Pacific Plate and the North American Plate. The Pacific Plate moves northwestward relative to the stable interior of the continent at an average rate of about 1.93 in/yr. (De Mets et al., 1990) roughly parallel with the coast of central California. The boundary between the two plates encompasses all of coastal California at the latitude of the San Francisco Bay (Page, 1990). The San Andreas fault accommodates less than half of the average total relative plate motion in the Bay Area with much of the remaining motion shared by such faults as the San Gregorio, Hayward, Calaveras, and Greenville (Fig. 12). Most of these faults have produced large historic earthquakes (Page, 1992).

Faults in the immediate vicinity of the PWTP include the Berryessa fault zone situated about 1000 feet to the northeast, the Crosley fault located about 600 feet to the northeast, and the Clayton fault located about 300 feet northeast of the PWTP (Coyle, 1984) (Figures 13 & 14). These faults are part of a group of transpressive faults that lie southwest of the Hayward fault in the foothills along the eastern margin of the valley floor. Transpressive faults exhibit both lateral (strike-slip) and compressive movements. In particular, the Berryessa, Crosley, and Clayton faults represent predominantly compressive movement that is the result of a slip junction between the strike-slip Calaveras and Hayward faults (WGNCEP, 1996; Petersen and others, 1966). The slip junction is thought to be the result of the transfer of continuous right lateral creep along the active southern trace of the Calaveras fault to right lateral creep along the active northern trace of the Hayward fault. The State of California and the U.S. Geological Survey have designated this group of transpressive faults as the "Southeast Extension of the Hayward fault" (WGNCEP, 1996; Petersen and others, 1996).

The Southeast Extension of the Hayward fault as delineated by the State of California is situated about 4500 feet northeast of the PWTP and the southern Calaveras fault is situated about 3 miles northeast of the PWTP. The southeast extension of the Hayward fault is thought to be capable of producing a magnitude 6.4 earthquake with a recurrence interval of 220 years. The principal trace of the southern section of the Hayward fault, prior to branching off as the southeast segment, is situated about 4 miles NW of the PWTP. The principal trace of the southern Hayward fault is thought to be capable of producing a magnitude 6.9 with a recurrence interval of 167 years. The southern Calaveras fault is capable of producing a magnitude 6.2 earthquake with a recurrence interval of 70 or 80 years, whereas the northern Calaveras fault is capable of producing a magnitude 6.9 earthquake with a recurrence interval of 146 years (Working Group on Northern California Earthquake Potential, 1996; Petersen and others, 1996).

The compressive component of movement along the local transpressive faults results in vertical deformation to produce steep slope gradients that contributes to mass wasting, defined as the down slope movement of rock and regolith near the earth's surface mainly due to the force of gravity. Consequently, much of the mountainous terrain in the vicinity of the site is covered with massive landslide complexes (Figure 13).

3.2 Local Geology

Bedrock underlying the foothills along the eastern boundary of the Santa Clara Valley consists of highly folded and faulted Jurassic-Cretaceous marine sedimentary rocks of the Berryessa Formation, and Miocene rocks of the Monterey Group and the Briones Formation. These rocks are in turn unconformably overlain by folded and poorly indurated, terrestrial clays, sands, and gravels of the Plio-Pleistocene Santa Clara Formation.

The following sections present only those geologic units specifically pertinent to the Penitencia Creek Landslide; namely the Santa Clara Formation and all other younger units.

3.2.1 Santa Clara Formation

The Santa Clara Formation consists of Plio-Pleistocene nonmarine sediments that were deposited along the flanks of newly emerging mountains of the Bay Area during an earlier period of mountain building. These sediments consist primarily of sands and gravels deposited by streams with lesser amounts of clays and silts deposited in lakes or as overbank flood deposits. The Santa Clara Formation has been subdivided into eight mappable lithofacies based on composition of conglomerate (gravel) clasts (Cummings, 1968, 1972). The term lithofacies is a geologic term applied to any sedimentary unit distinguished from other adjacent subdivisions on the basis of noteworthy geologic characteristics. Typical clasts found in the Santa Clara Formation include siliceous clasts, graywacke, greenstone, and chert, with rare serpentinite and serpentinized gabbros. The greenstone includes altered aphanitic basalt and basalt breccia.

The assemblage of Santa Clara Formation sediments are geologically young, semi-lithified, with properties between rock and soil. Claystone units that are unweathered and unsheared possess shear strengths higher than soil, but significantly lower than most rock. In addition, the claystones are typically overconsolidated, indicating that compared to the present they have been subjected to significantly higher overburden pressures and/or desiccation in the recent geologic past.

According to mapping by John Coyle (1984), the Penitencia Creek Landslide has formed on the Santa Clara Formation, which is prone to landsliding throughout its extent along the eastern and western margins of the southern San Francisco Bay Area. Nelson (1985) compiled subsurface borehole and trench data from 10 development sites prone to landsliding in the Saratoga foothills Area of California, to correlate the occurrence of claystone with landsliding. Overall, claystone lithology was encountered in roughly 75 percent of the borings and trenches at the 10 development sites. Significantly, there was a 90 percent occurrence of claystone encountered in actual landslide masses.

3.2.2 Older Alluvial Deposits

Information from drill holes suggests that older alluvial deposits underlie much of the southwestern portion of the Penitencia Creek Landslide. These sediments appear to have been deposited in the ancestral Santa Clara Valley. As discussed in more detail in Chapter 7, at some point in the geologic past the Penitencia Creek Landslide appears to have moved onto the floor of the ancestral Santa Clara Valley. Evidence for this includes the following: 1) the presence of a nearly horizontal shear surface beneath the southwestern portion of the landslide, 2) an anomalous orientation of the shear surface indicating the presence of an ancestral stream channel of Dutard Creek, and 3) the presence of alluvial deposits encountered in inclinometer borings beneath the landslide.

The most detailed lithologic boring logs on record at the District from the PWTP are from work by Woodward-Clyde Consultants (WCC, 1997), who logged inclinometer borings I-26 and I-27, and from work by DWR (2000) who recently logged inclinometer borings DWR-6, DWR-7, and DWR-8 near the Terminal Reservoir. Logs of inclinometer borings prior to the 1997 WCC work were less detailed because only drill cuttings were logged. Logging of cuttings is not a reliable method for making detailed observations of subsurface conditions because drilling mixes materials from various geologic horizons as the cuttings come up the boring. The logging by WCC and DWR included rock coring, which permits direct observations of intact cored samples. In particular, DWR's work included continuous coring in each boring to depths of about 100 feet (DWR-7) and 300 feet (DWR-6). In this manner, DWR Senior Geologist Mr. Rob Berry was able to directly observe the entire section penetrated by the inclinometer borings.

Evidence from subsurface logs and laboratory testing indicates a significant contrast between predominantly fine-grained clayey and silty soils (with varying amounts of sand and gravel) of the landslide mass and coarse-grained sandy soils defined as the older alluvium (with varying amounts of clay and silt) directly beneath the mass. In boring I-26, located about 160 feet east of the Clearwell, WCC (1997) logged various highly plastic clays and silts typical of the landslide mass to a depth of about 203 feet. This depth approximately corresponds with the bottom of the landslide. Subsequent inclinometer probe data indicates that the primary landslide surface is present at a depth of about 190 feet. From 203 feet to the bottom of the boring at 230 feet, WCC (1997) logged poorly graded sand. The presence of sandy soils is usually suggestive of alluvial (stream) deposits. Notes on the logs indicate that between 203 feet and 230 feet drilling fluids were lost to the boring. Loss of fluid correlates well with the presence of relatively permeable poorly graded sands. Similar geologic conditions were encountered in boring DWR-6, located near the Terminal Reservoir. DWR (2000) logged highly plastic clays and silts to a depth of about 193 feet. This correlates to the expected depth to the primary landslide surface. From 193 feet to the bottom of the boring at 308 feet (total of 115 feet) DWR logged predominantly clayey sand with gravel with localized layers of fat clay with sand, which were less than about 2 feet thick.

Laboratory index testing was performed by DWR on seven composite soil samples collected from the core of two inclinometer borings (DWR-6 and DWR-8) at selective depths. Five of the composite samples (S-1, S-2, S-5, S-6, and S-7) contained representative soils from various depths within the landslide mass. Two of the composite samples (S-3 and S-4) contained representative soils within the lower older alluvium. Index testing consisted of Atterberg limits and gradation analysis. All of the landslide-derived samples were laboratory classified as highly plastic clay (CH) with percent fines (defined as the percent passing #200 sieve, 0.075mm) that ranged from about 50 to 75 percent. The two older alluvium samples were laboratory classified as poorly graded gravel with clay and sand (GP-GC) and clayey sand with gravel (SC), with percent fines that ranged from about 8 to 28 percent. Figure 19 shows a photograph of the core of clayey sand with gravel from DWR's report (2000), which we have identified as older alluvium. The presence of sufficient coarse-grained material, as evident in the photograph, suggests these soils are significantly more permeable than the typical clayey soils of the Santa Clara Formation-derived landslide.

3.2.3 Younger Alluvial Deposits

Younger alluvial deposits make up the floor of the valley at the toe of the landslide mass. These deposits consist of gravels, sands, silts, and clays that formed from erosion and depositional processes of slopes on adjacent hills and from stream channels.

3.2.4 Toe Terrace Deposits

As discussed in more detail in Chapter 7, various geologic evidence indicates the presence of a particular landform at the landslide toe which we have termed "toe terrace deposits." These terraces, or step-like features, are located between the valley floor and the toe of Penitencia Creek Landslide (Figure 17). District GPS survey monuments across both the toe of the landslide and the toe terrace deposits strongly indicate that while the landslide is obviously moving, the toe terrace deposits are not. It is not clear exactly how the landslide toe terrace developed. However, in Chapter 7 we postulate that the terraces form due to passive shearing deformation that propagates outward (downslope) from the landslide toe until a continuous surface of failure extends to the ground surface and the surface heaves. This process would be similar to bearing-capacity failure beneath a foundation.

3.3 Geologic Structure

Geologic mapping by Crittenden (1951) and Coyle (1984) indicate that all geologic units on the western flank of the Diablo Range in the vicinity of the PWTP have been tilted to the east with intervening faults as shown in Figure 15. Extensive folding of Tertiary-aged and older rocks has produced a synclinal fold (U-shaped) known as the Turlarcitos Syncline, which is situated about 2 miles east of the site (Crittenden, 1951). A geologic map and geologic cross section of the site is presented in Figures 16 & 17. A more detailed discussion of the possible geologic factors that contributed to the slope evolution of the Penitencia Creek Landslide area is further discussed in Section 3.6.

3.4 Seismic Considerations

Faults that have the possibility of causing significant ground shaking at the project site are listed on Table 1 and shown in Figure 12 which represents a computer generated Regional Fault Location and Seismicity Map. As shown on Table 1, 26 seismogenic sources (faults) are shown to exist within a distance of 90 mi. that could possibly impact the site. There are five pieces of site-specific data shown for each fault, including: 1) the fault length; 2) the distance and direction of the fault from the PWTP site; 3) the estimated maximum peak moment magnitude earthquake expected for each fault; 4) the estimated peak ground acceleration that would be generated at the PWTP site under the maximum peak moment magnitude earthquake; and 5) the estimated recurrence interval (years) between historic peak magnitude earthquakes for each fault. The maximum earthquake is either based on historic data or is the maximum earthquake that appears capable of occurring under the present tectonic framework (Working Group on Northern California Earthquake Potential, 1996; Petersen and others, 1996). The estimated mean peak ground acceleration was computed using the earthquake attenuation models and methods discussed in Abrahamson & Silva (1997).

The Working Group on Northern California Earthquake Potential (1996) and Petersen and others (1996) has identified the faults listed on Table 1 as capable of producing earthquakes larger than moment magnitude 6 in the vicinity of PWTP. Coyle (1984) has mapped two additional faults of unknown activity that could significant impact the site. The Clayton and the Crosley faults are part of the Berryessa fault complex that essentially underlies the site. These faults, along with all faults catalogued on Table 1 are also shown in Figures 13 and 16, respectively.

Recent data from the U.S. Geological Survey (Working Group on California Earthquake Probabilities, 1999) indicates there is a 70 % chance for at least one earthquake of magnitude 6.7 or greater in the San Francisco Bay Area in the next 30 years. Earthquakes of this magnitude could occur on the following faults or fault segments closest to the PWTP, the San Gregorio fault, the San Francisco Peninsula segment of the San Andreas fault, the Calaveras fault, the northern and southern portions of the Hayward fault, and the Rodgers Creek fault (Figure 12). The calculated probability of occurrence of a magnitude 6.7 in the next thirty years (2001 to 2031) is 32 % for combined fault slip on the Rodgers Creek and Hayward faults, 21 % on the San Andreas fault, 18 % on the Calaveras fault, 10 % on the San Gregorio fault, 6 % on the Greenville fault, 6 % on the Concord-Green Valley fault, and 4 % on the Mt. Diablo thrust fault, for an aggregate probability of 70 % (Figure 12).

3.5 Historical Earthquakes

The April 24, 1984 Morgan Hill (M=6.2) and October 17, 1989 Loma Prieta (M=6.9) earthquakes (hereafter termed the Morgan Hill and Loma Prieta earthquakes, respectively) caused measurable accelerated creep of the Penitencia Creek Landslide. As shown on Figure 12, the Morgan Hill earthquake occurred on the Calaveras fault with a magnitude of 6.2 and focal depth of 5 miles. The Loma Prieta earthquake occurred on the San Andreas fault system with a magnitude of 6.9 and a focal depth of 12 miles. The Penitencia Creek

Table 1
Major Faults and Seismic Considerations – Penitencia Water Treatment Plant

Fault name & Fault type	Length km (miles)	Distance & direction from site km (miles)	Historic* Moment Peak Magnitude (M_w)	Estimated** Peak Ground Acceleration (g)	Effective Recurrence Interval, yr
Hayward, Southeast Ext (R & RL)	26 (16)	1.4 (0.9) to NW	6.4	0.69	220
Southern Calaveras (RL)	29 (18)	5 (3) to NW	6.2	0.43	33
Hayward South (RL)	43 (27)	6 (4) to NW	6.9	0.52	167
Northern Calaveras (RL)	52 (32)	6 (4) to NW	6.8	0.50	146
Monte Vista-Shannon (R)	41 (25)	20 (12) to NE	6.8	0.19	2,410
Greenville (RL)	73 (45)	28 (17) to N-NW	6.9	0.14	521
San Andreas Peninsula (RL)	88 (55)	28 (17) to SW	7.1	0.15	400
San Andreas 1906 rupture (RL)	470 (292)	28 (17) to SW	7.9	0.21	210
San Andreas SC Mts (RL)	37 (23)	29 (18) to SW	7.0	0.14	400
Sargent (R) & (RL)	53 (33)	30 (19) to SW	6.8	0.12	1,200
Zayante-Vergeles (R & RL)	56 (35)	36 (22) to SW	6.8	0.10	8,821
Great Valley 6 (R)	45 (28)	45 (28) N-NW	6.7	0.08	622
Hayward North (RL)	43 (27)	45 (28) to N-NW	6.9	0.08	167
Great Valley 7 (R)	45 (28)	46 (29) to NE	6.7	0.07	622
San Gregorio (R & RL)	129 (80)	50 (31) to SW	7.3	0.09	400
Ortigalita (RL)	66 (41)	51 (32) to NE	6.9	0.07	1,153
Concord-Green Valley (RL)	66 (41)	55 (34) to NW	6.9	0.06	176
Monterey Bay - Tularcitos (R & RL)	83 (52)	59 (37) to S-SW	7.1	0.06	2,600
Great Valley 8 (R)	41 (25)	60 (37) to E	6.6	0.04	483
Quien Sabe (RL)	22 (14)	67 (42) to SE	6.4	0.03	647
San Gregorio – Sur (RL)	79 (49)	72 (45) to SW	7.0	0.05	411
Great Valley 5 (R)	28 (17)	76 (47) to SE	6.5	0.03	622
Rinconada (RL)	190 (118)	80 (50) to S-SE	7.3	0.06	1,764
San Andreas (North Coast) (RL)	322 (200)	81 (50) to NW	7.6	0.07	N/A
Great Valley 9 (R)	39 (24)	82 (51) to E-SE	6.6	0.03	508
Rodgers Creek (RL)	63 (39)	88 (55) to NW	7.0	0.04	222

Notes:

1) * The maximum earthquake (based on historic data) that appears capable of occurring under the presently known tectonic framework (Working Group on Northern California Earthquake Potential, 1996 and Probabilistic Seismic Hazard Assessment for the State of California, 1996, CDMG Open-File Report 96-08).
2) ** The estimated peak ground acceleration value is determined using the earthquake attenuation relationship by Abrahamson and Silva (1995).
3) (RL) = right lateral strike slip fault, (R) = reverse fault (File: W:\geology\penitencia creek landslide\final report\table1)

Landslide is located about 9 miles northwest of the Morgan Hill earthquake epicenter and about 25 miles northeast of the Loma Prieta earthquake epicenter. Although no strong motion accelerometer records were obtained on the Penitencia Creek landslide, using the previously referenced attenuation models (Abrahamson and Silva, 1997) we estimate that the peak ground acceleration at the site was 0.18 g for the Morgan Hill earthquake and 0.10 g for the Loma Prieta earthquake.

Additional noteworthy earthquakes in the vicinity of the PWTP are shown on Figure 12. These include the August 6, 1979 Coyote Lake earthquake that had a local magnitude of 5.8, the May 2, 1983 Coalinga earthquake that had a magnitude of 6.7, and the March 31, 1986 Mt. Lewis earthquake that had a magnitude of 5.7. The Penitencia Creek Landslide is located about 28 miles northwest of the Coyote Lake earthquake, about 117 miles northwest of the Coalinga earthquakes, and about 10 miles southwest of the Mt. Lewis earthquake. We estimate that the peak ground acceleration at the site was 0.03 g for the Coyote Lake earthquake, 0.02 g for the Coalinga earthquake, and 0.10 g for the Mt. Lewis earthquake.

3.6 Slope Evolution - Local Mountain Building, Deposition of Ancestral Santa Clara Formation, and Landslide Development

The following is a brief summary of possible geologic factors that contributed to slope evolution, including initiation of local mountain building, deposition of the Santa Clara Formation, and general development of the Penitencia Creek Landslide. A detailed discussion of landsliding mechanisms is presented in Chapter 7. This summary was significantly improved after collaborative discussions with three geologist experts; including, David Andersen, San Jose State University, and Carl Wentworth and John Tinsley, III with the U.S. Geological Survey. Although these prominent engineering geologists provided useful information to the authors dealing with discussion of possible geologic history and factors that contributed to the development of the Penitencia Creek landslide, the authors assume complete responsibility for the discussion that follows.

Figure 18 consists of 6 block schematic diagrams that depict a hypothesized slope evolution in the Penitencia Creek Area, as described below. Our slope evolution model is based on the work of Vanderhurst (1981) and Vanderhurst and others (1982) in the Saratoga Foothills Area. It is reasonable to apply these earlier findings to our discussion of the Penitencia Creek Area because the engineering geologic characteristics of the Santa Clara Formation claystone units at both Penitencia Creek and Saratoga Foothills Areas are quite similar. These similarities are discussed in more detail in Chapter 4.

3.6.1 Stage 1 – New Tectonic Regime (~4 Million Years Ago) Deposition of Lower Ancestral Santa Clara Sediments by Braided Streams and Lakes

As shown in Figure 18, a new tectonic regime is thought to have begun along the ancestral San Andreas fault system about 4 million years ago (Ma), which involved predominantly right lateral strike-slip motion with an imposed fault normal component. Prior to that time, the motion consisted of simple right lateral strike slip. The new tectonic regime with a fault normal component marked the beginning of an initial phase of mountain building of the ancestral Santa Cruz Mountains and Diablo Range and subsequent deposition of ancestral Santa Clara Formation sediments.

Deposition of all eight lithofacies of the Santa Clara Formation was initiated with the change in the tectonic regime. According to Andersen (personal communication, 2000) and Vanderhurst (1981), the Penitencia Creek Area was probably a site of erosion during deposition of the Arastradero Lithofacies along the western edge of Santa Clara Valley. This assumption is based on geologic evidence, which suggests that the Penitencia Creek Area was a possible source for gravels in the Arastradero Lithofacies in the Saratoga/Cupertino Area. The Arastradero is the lowermost and oldest of the eight lithofacies, with a probable age of late Pliocene (~2 to 4 Ma). Consequently, the Penitencia Creek area likely became an area of deposition of Santa Clara Formation sediments later, during the late Pliocene to early Pleistocene (<2 Ma).

In the Penitencia Creek Area, the ancestral claystone sediments were probably deposited in environments similar to the Arastradero Lithofacies in the Saratoga Foothills Area (Vanderhurst, 1981). As the Diablo Range mountains rose, the newly formed basin at the western flank of the mountains filled with coarse-grained sediments (predominantly sand and gravel) deposited by braided streams and fine-grained silts and clays deposited in lakes (lacustrine). The lacustrine deposits eventually developed into overconsolidated claystone units involved in landsliding.

3.6.2 Stage 2 – Deposition of Upper Ancestral Santa Clara Sediments by Alluvial Fans

According to Vanderhurst and others (1982), from the time of deposition of the Arastradero Lithofacies to deposition of older overlying lithofacies (i.e. Stevens Creek Lithofacies), the sedimentary environment shifted from a braided-stream system to an alluvial fan-dominated system. This probably occurred in the Penitencia Creek Area during early-middle Pleistocene (~1 to 2 Ma).

As a result of range-front thrust fault movement, the rising ancestral Diablo Range mountains produced relatively steep west-facing slopes. This marked the beginning of deposition of predominantly coarse-grained sediments in coalescing alluvial fans that flanked the western slopes of the Diablo Range. These alluvial fan materials rapidly infilled the adjacent ancestral Santa Clara Valley and buried the predominantly finer grained underlying lacustrine and braided stream deposits. Erosion of the mountain range resulted in a fairly thick sequence of gravel deposits. Consequently, most sedimentologists think of the Santa Clara Formation as being composed predominantly of coarse-grained sands and gravels. To our knowledge, no one has attempted to quantify the ratio of coarse-grained to fine-grained materials in this formation.

3.6.3 Stage 3 – Completion of Ancestral Santa Clara Sediment Deposition and Beginning of Consolidation

Deposition of the alluvial fan sediments of the ancestral Santa Clara Formation in the Penitencia Creek Area was probably completed by middle to early-late Pleistocene (0.5 to 1

Ma). In the Saratoga Foothills Area the Santa Clara Formation obtained an estimated thickness of about 1,500 feet (Vanderhurst, 1981; Andersen, 2000). No detailed analysis of total formation thickness, however, has been performed in the vicinity of Penitencia Creek (Andersen, 2000). The maximum thickness of the Santa Clara Formation at Penitencia Creek is estimated as ranging from 1.5 to 2.0 times the present thickness (Wentworth, 2000). Based on local subsurface data, the Santa Clara Formation is at least 300 feet thick, which includes 270 feet of landslide material. Consequently, the formation was probably at least 600 feet thick at the completion of stage 3. This amount of sediment, or possibly thicker, began consolidating the underlying lacustrine deposits.

3.6.4 Stage 4 - Second Phase of Diablo Range Mountain Building, Folding and Erosion of Santa Clara Formation Sediments

By late Pleistocene (0.1 to 0.5 Ma), Santa Clara Formation sediments were folded during a second phase of mountain building, which produced the present Diablo Range. Folding and uplift probably coincided with the development of imbricate thrust faulting along ancestral Berryessa, Crosley, and Clayton faults. Uplift along these imbricate thrust faults resulted in erosion of the upper proximal fan deposits and portions of the underlying braided streams/lacustrine deposits.

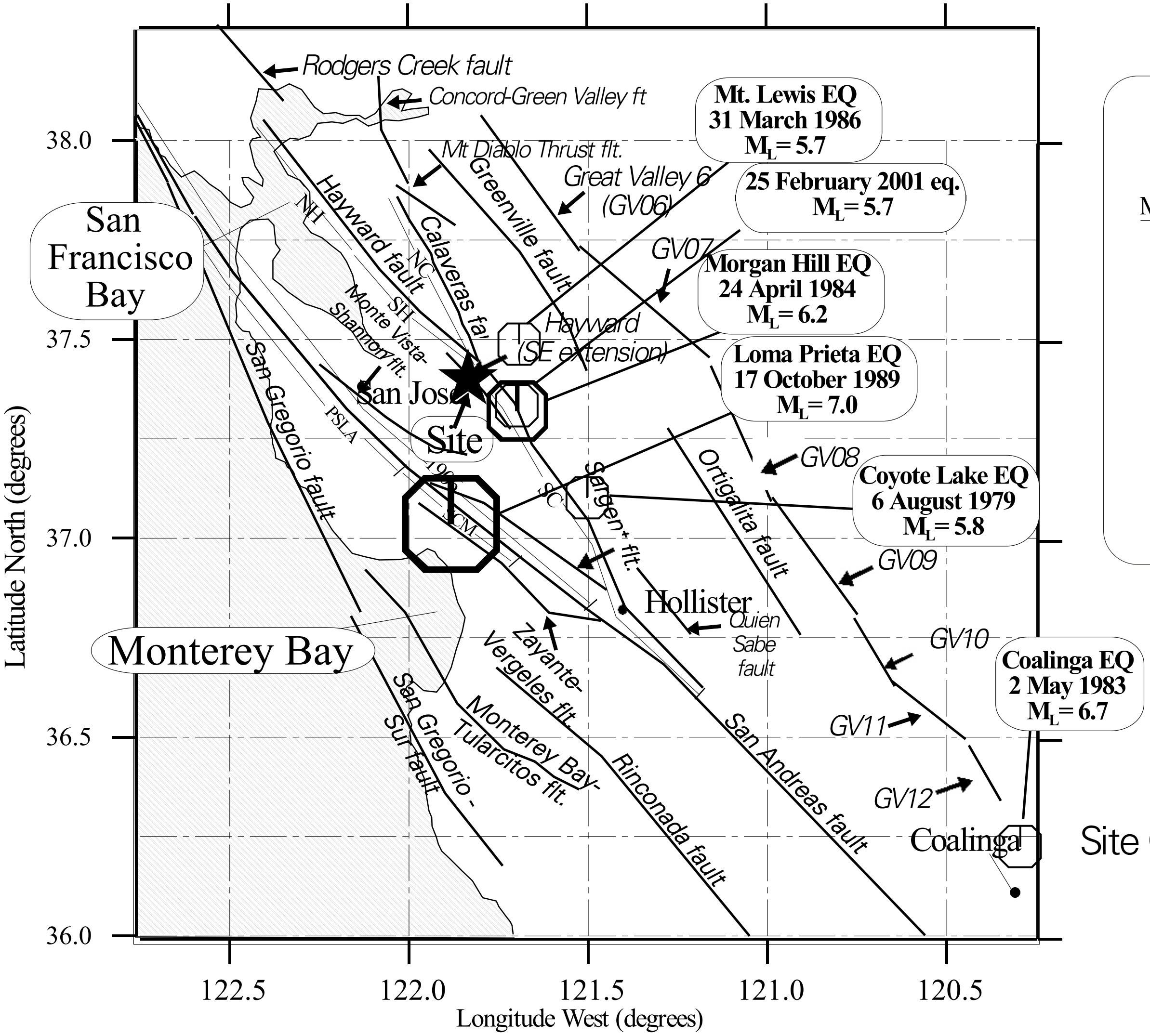
The present topography of the Santa Cruz Mountains and Diablo Range surrounding the south Bay Area has not changed significantly during the last ~100,000 years (Tinsley, 2000).

3.6.5 Stage 5 – Maximum Glacial Period Produced Wetter Climate - ~18,000 to 20,000 Years Ago

About 18,000 to 20,000 years ago, during the last maximum glacial period, the Bay Area possessed a wetter climate, much like the present one in Oregon and Washington (Tinsley, 2000). Consequently, it is believed that many of the large ancient "Quaternary" landslides like the Penitencia Creek Landslide bordering the Santa Clara Valley formed during that wetter period. The antiquity of these large Quaternary landslides is evident by the presence of a subdued and rounded topography with superimposed incised erosional features.

3.6.6 Stage 6 - Present Conditions Marked by Long Term Landslide Creep

From about 20,000 years ago to the present, the Penitencia Creek Landslide has developed or evolved into a slowly creeping landslide mass. An interesting question to ask is …if the Penitencia Creek Landslide Complex began moving about 20,000 years ago when the climate was significantly wetter than it is today, why does the landslide continue to move in today's drier climate? The answer to this question has to do with the development of the very low residual shear strength within the Santa Clara formation, which is discussed in more detail in Chapter 4. A more detailed discussion of how the rate of landslide movement has continued to decay since the original landslide inception is presented in Chapter 7.



Explanation

Fault segment abbrivations/ Probability of Magnitude 6.7 or greater occuring in next 30 years

NC = Northern Calaveras
SC = Southern Calaveras/ 18%
PSLA = Peninsula Segment of the San Andreas/ 21%
SCM = Santa Cruz Mts.
1906 = 1906 EQ
NH = Northern Hayward/ 32%
SC = Southern Hayward
GV = Great Valley
Concord-Green Valley/ 6%
San Gregorio/ 10%
Mt. Diablo thrust fault/ 4%
Greenville/ 6%

Site Coordinates: Latitude: 37.3992 N
Longitude: 121.8355 W

Notes: 1) File W:\geology\penitencia creek landslide\figures\fig12.grf

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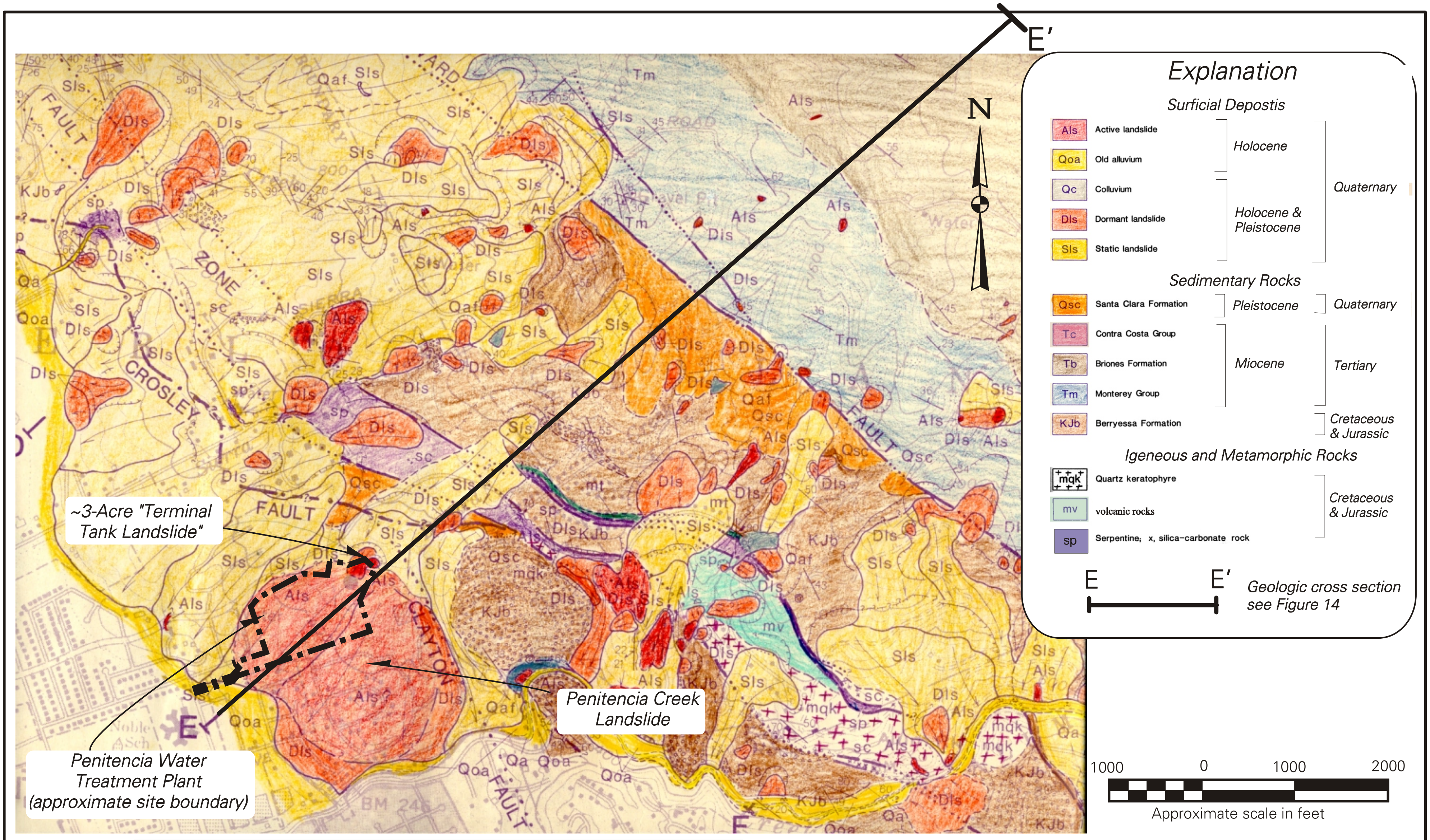
Santa Clara Valley Water District

Regional Fault Location and Seismicity Map

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=20'

FIGURE 12



Notes: 1) Ref: Coyle, 1984, southeast portion of Plate 1, Geologic Map of the San Jose-Milpitas Foothills Area, Santa Clara County, California (unpublished M.S. Thesis)
2) File: W:\geology\penitencia creek landslide\figures\fig13.cdr

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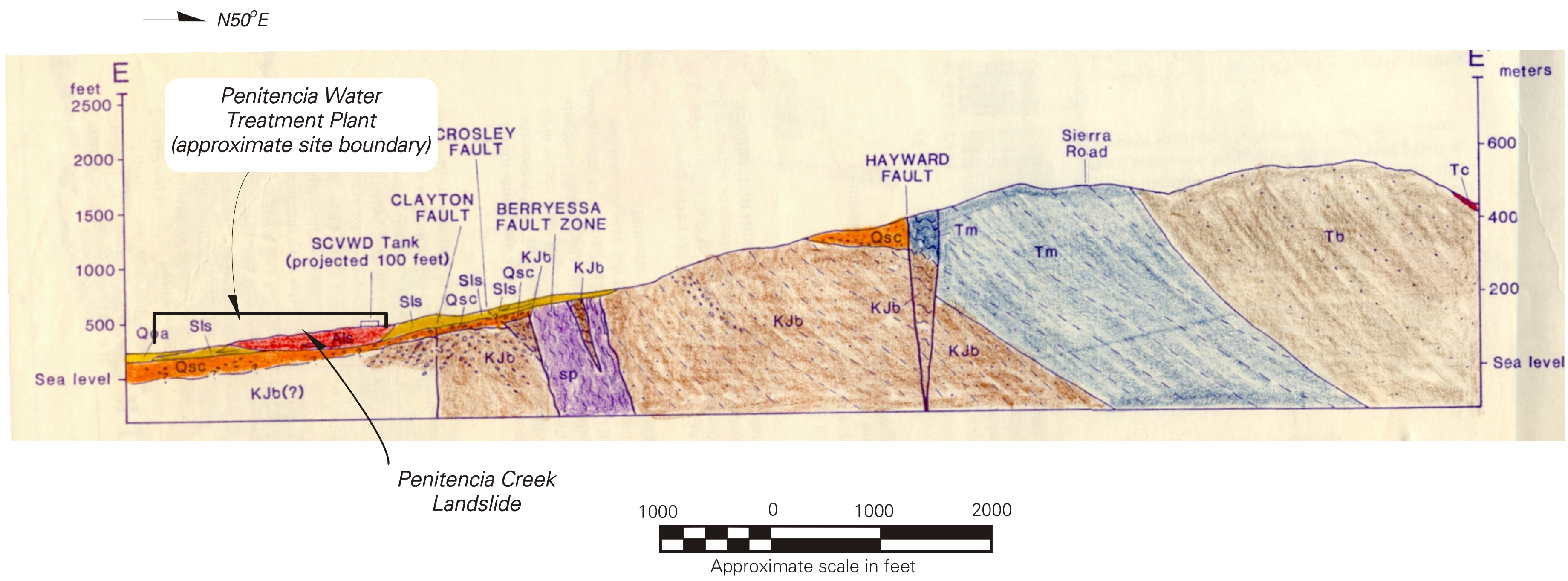
Santa Clara Valley Water District

Geologic Map - John M. Coyle, 1984

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1" = 1000'

FIGURE
13



Notes: 1) see Figure 13 for explanation and location of section
2) Ref: Coyle, 1984, Plate 2, Geologic Cross Section of the San Jose-Milpitas Foothills Area, Santa Clara County, California (unpublished M.S. Thesis)
3) File: W:\geology\penitencia creek landslide\figures\fig14.cdr

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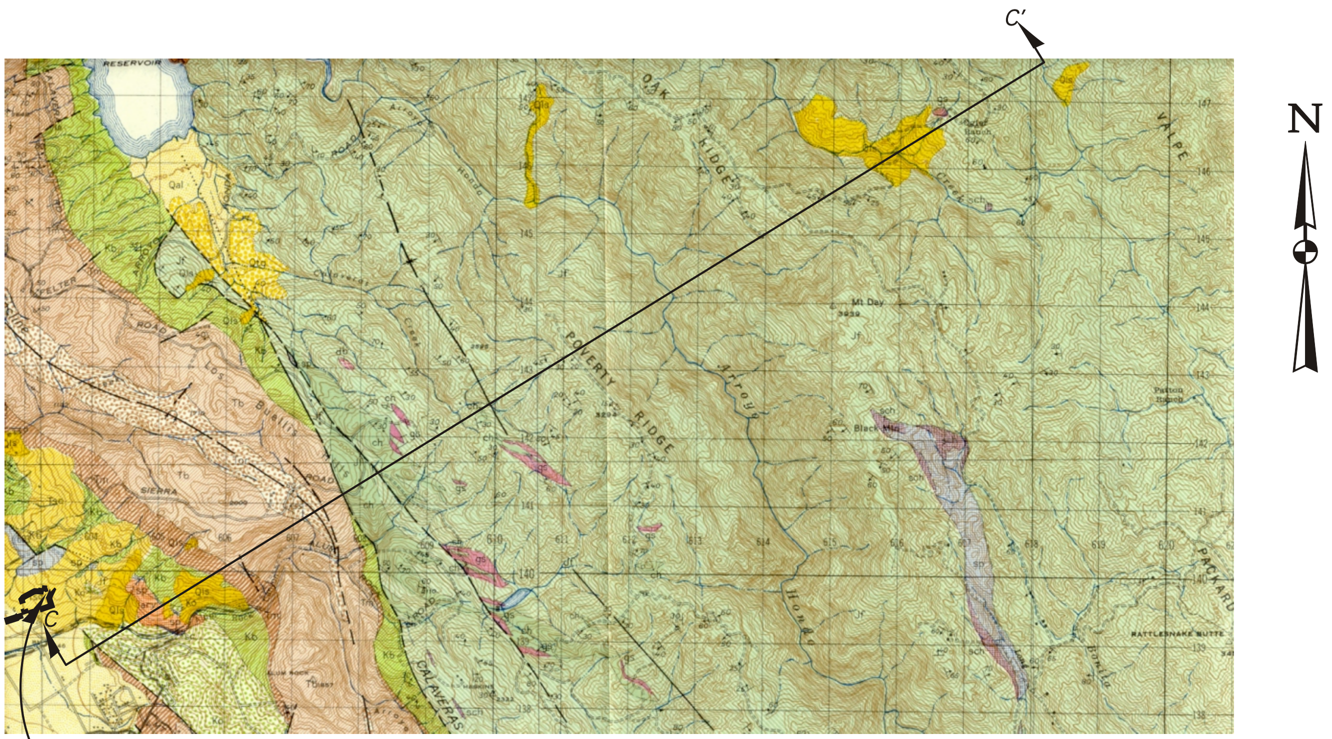
Santa Clara Valley Water District

Geologic Cross Section
John M. Coyle, 1984

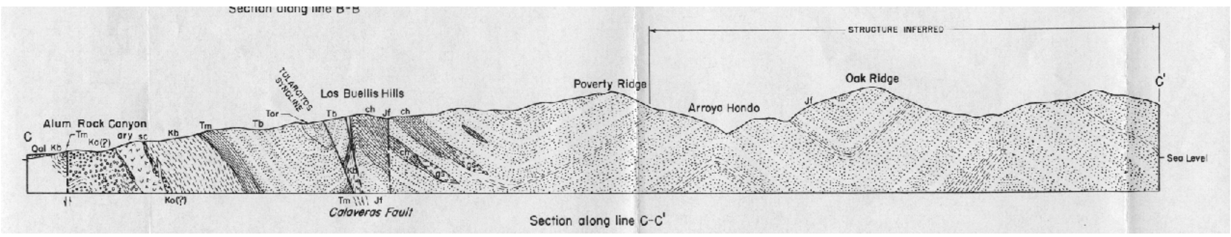
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1" = 1000'

FIGURE 14

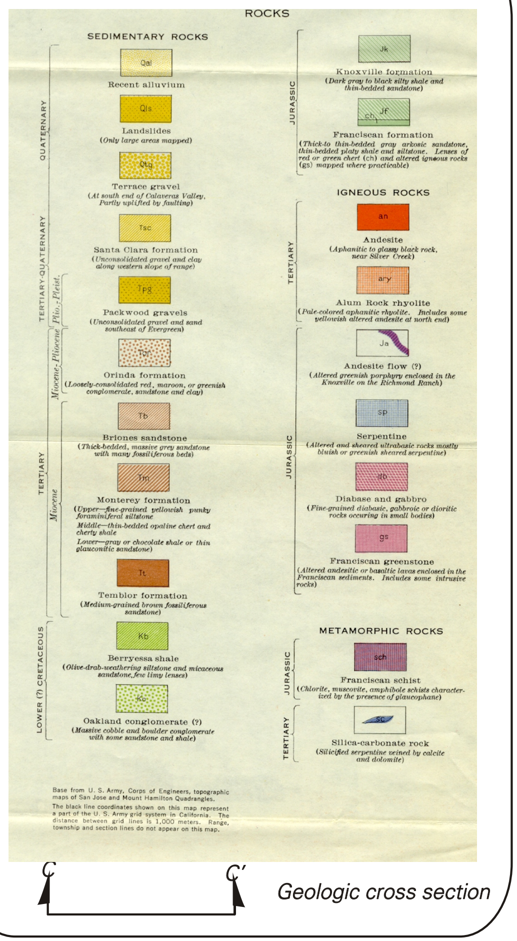


Penitencia Water Treatment Plant (approximate site boundary)



Section along line C-C'

Explanation



ROCKS

SEDIMENTARY ROCKS

QUATERNARY

- Qal — Recent alluvium
- Qls — Landslides (*Only large areas mapped*)
- Qtg — Terrace gravel (*At south end of Calaveras Valley, Partly uplifted by faulting*)

TERTIARY-QUATERNARY

- Tsc — Santa Clara formation (*Unconsolidated gravel and clay along western slope of range*)
- *Plio.-Pleist.* Tpg — Packwood gravels (*Unconsolidated gravel and sand southeast of Evergreen*)

TERTIARY

- *Miocene-Pliocene* Tor — Orinda formation (*Loosely-consolidated red, maroon, or greenish conglomerate, sandstone and clay*)
- *Miocene* Tb — Briones sandstone (*Thick-bedded, massive grey sandstone with many fossiliferous beds*)
- Tm — Monterey formation (*Upper—fine-grained yellowish punky foraminiferal siltstone*
  Middle—thin-bedded opaline chert and cherty shale
  Lower—gray or chocolate shale or thin glauconitic sandstone)
- Tt — Temblor formation (*Medium-grained brown fossiliferous sandstone*)

LOWER (?) CRETACEOUS

- Kb — Berryessa shale (*Olive-drab-weathering siltstone and micaceous sandstone, few limy lenses*)
- Ko — Oakland conglomerate (?) (*Massive cobble and boulder conglomerate with some sandstone and shale*)

JURASSIC

- Jk — Knoxville formation (*Dark gray to black silty shale and thin-bedded sandstone*)
- Jf, ch — Franciscan formation (*Thick-to thin-bedded gray arkosic sandstone, thin-bedded platy shale and siltstone. Lenses of red or green chert* (ch) *and altered igneous rocks* (gs) *mapped where practicable*)

IGNEOUS ROCKS

TERTIARY

- an — Andesite (*Aphanitic to glassy black rock, near Silver Creek*)
- ary — Alum Rock rhyolite (*Pale-colored aphanitic rhyolite. Includes some yellowish altered andesite at north end*)

JURASSIC

- Ja — Andesite flow (?) (*Altered greenish porphyry enclosed in the Knoxville on the Richmond Ranch*)
- sp — Serpentine (*Altered and sheared ultrabasic rocks mostly bluish or greenish sheared serpentine*)
- db — Diabase and gabbro (*Fine-grained diabasic, gabbroic or dioritic rocks occuring in small bodies*)
- gs — Franciscan greenstone (*Altered andesitic or basaltic lavas enclosed in the Franciscan sediments. Includes some intrusive rocks*)

METAMORPHIC ROCKS

JURASSIC

- sch — Franciscan schist (*Chlorite, muscovite, amphibole schists characterized by the presence of glaucophane*)

TERTIARY

- sc — Silica-carbonate rock (*Silicified serpentine veined by calcite and dolomite*)

Base from U. S. Army, Corps of Engineers, topographic maps of San Jose and Mount Hamilton Quadrangles.

The black line coordinates shown on this map represent a part of the U. S. Army grid system in California. The distance between grid lines is 1,000 meters. Range, township and section lines do not appear on this map.

C — C' Geologic cross section

Approximate scale in miles

Notes: 1) Ref: M.D. Crittenden, Jr.,1951, Geology of the San Jose-Mount Hamilton Area, California northern portion of Plate 1, Geologic Map of the San Jose-Mount Hamilton Area, California, scale 1:62500, and Plate 3, Geologic Sections through the San Jose-Mount Hamilton Area, California, CDMG Bulletin 157,
2) File: W:\geology\penitencia creek landslide\figures\fig15.cdr

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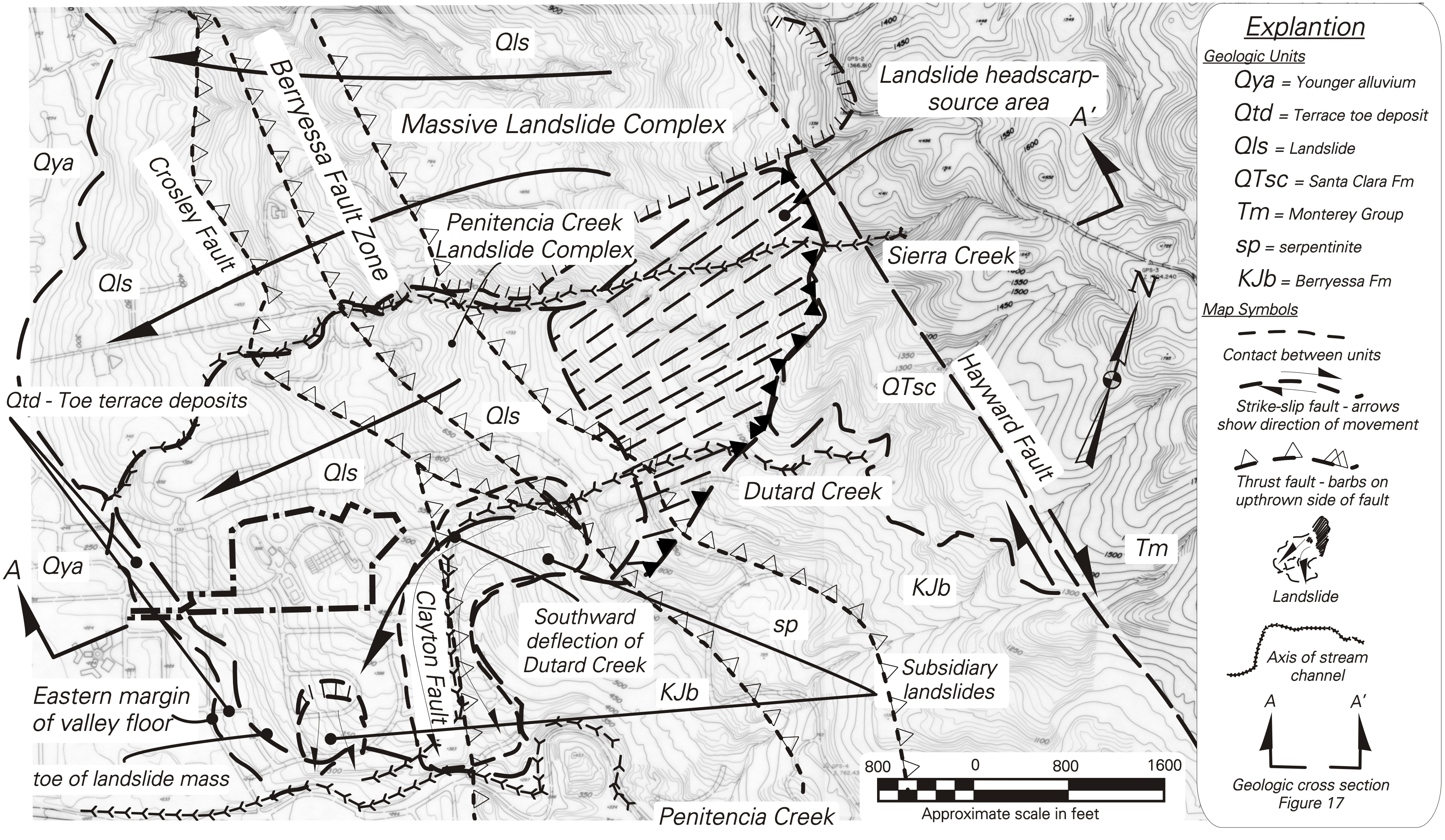
Geologic Map and Cross Section
M.D. Crittenden, 1951

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1" = 1 mi

FIGURE 15



Notes:
1) Date of photography: 8/10/89
2) Flown by Aero-Geodetic Corp.
3) Geologic mapping modified after John M. Coyle, 1984
4) File: W:\geology\penitencia creek landslide\figures\fig16.cdr

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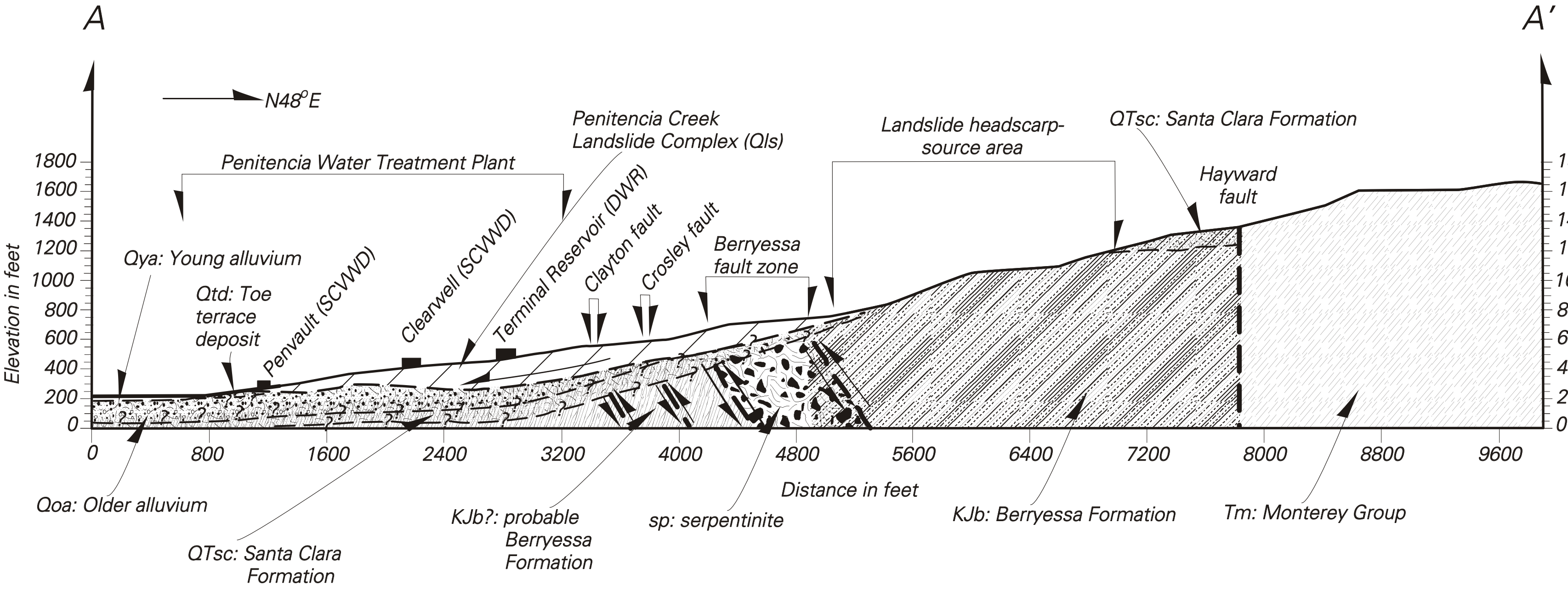
Santa Clara Valley Water District

Generalized Geologic Site Map

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=800'

FIGURE
16



Note: 1) modified after Coyle 91984) see Figure 16 for explanation and location of section
2) dashed landslide surface within area of water treatment plant based on actual inclinometer data; dashed and queried where extrapolated
3) see Figure 16 for explanation and location of section
4) File: W:\geology\nelson\penitencia creek landslide\figures\fig17.cdr

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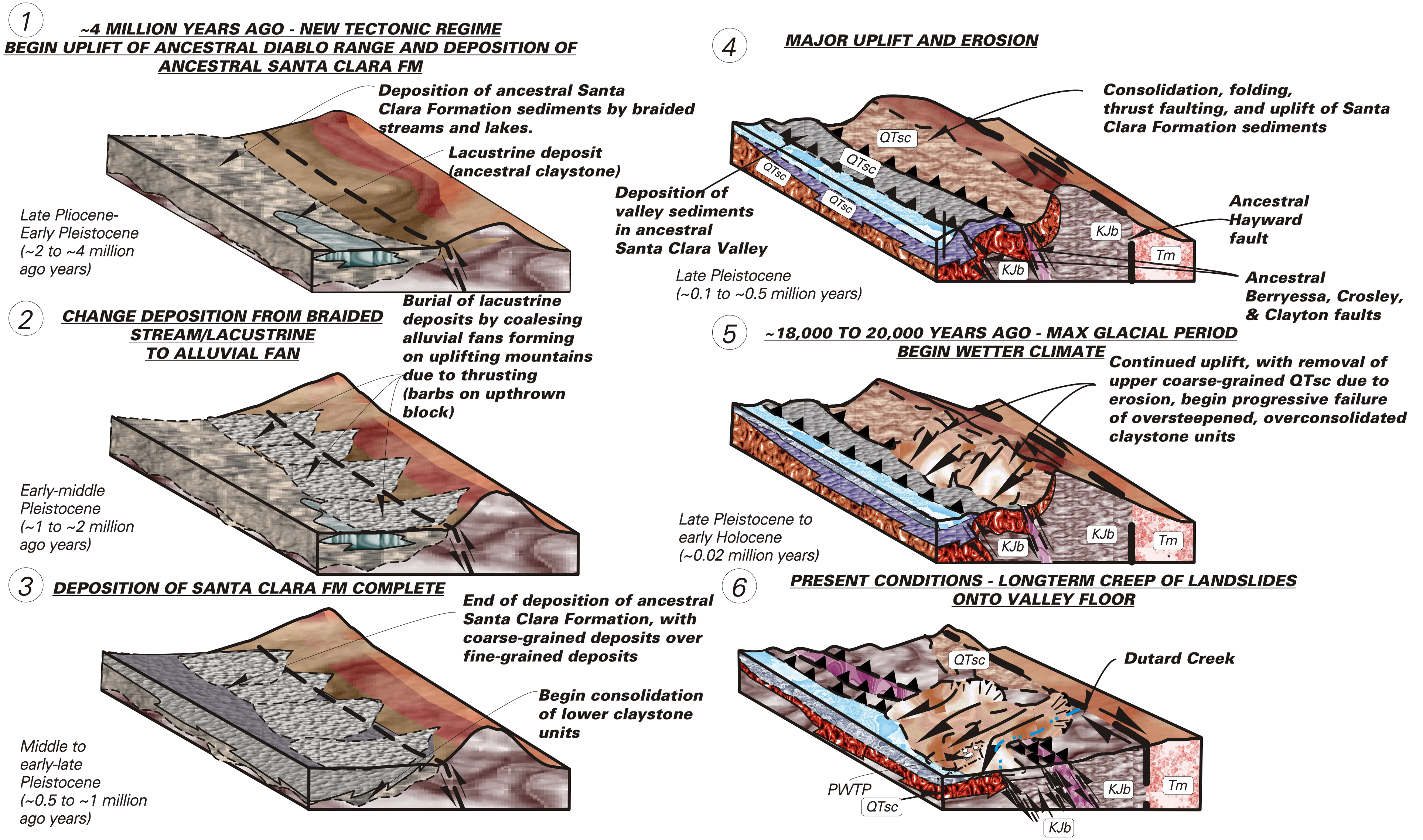
Santa Clara Valley Water District

Geologic Cross Section A-A′
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=800′
FIGURE
17


1
~4 MILLION YEARS AGO - NEW TECTONIC REGIME
BEGIN UPLIFT OF ANCESTRAL DIABLO RANGE AND DEPOSITION OF ANCESTRAL SANTA CLARA FM
Deposition of ancestral Santa Clara Formation sediments by braided streams and lakes.
Lacustrine deposit (ancestral claystone)
Late Pliocene-Early Pleistocene (~2 to ~4 million ago years)
2
CHANGE DEPOSITION FROM BRAIDED STREAM/LACUSTRINE TO ALLUVIAL FAN
Burial of lacustrine deposits by coalesing alluvial fans forming on uplifting mountains due to thrusting (barbs on upthrown block)
Early-middle Pleistocene (~1 to ~2 million ago years)
3
DEPOSITION OF SANTA CLARA FM COMPLETE
End of deposition of ancestral Santa Clara Formation, with coarse-grained deposits over fine-grained deposits
Begin consolidation of lower claystone units
Middle to early-late Pleistocene (~0.5 to ~1 million ago years)
4
MAJOR UPLIFT AND EROSION
Consolidation, folding, thrust faulting, and uplift of Santa Clara Formation sediments
QTsc
QTsc
QTsc
QTsc
Deposition of valley sediments in ancestral Santa Clara Valley
KJb
KJb
Tm
Ancestral Hayward fault
Ancestral Berryessa, Crosley, & Clayton faults
Late Pleistocene (~0.1 to ~0.5 million years)
5
~18,000 TO 20,000 YEARS AGO - MAX GLACIAL PERIOD
BEGIN WETTER CLIMATE
Continued uplift, with removal of upper coarse-grained QTsc due to erosion, begin progressive failure of oversteepened, overconsolidated claystone units
KJb
KJb
Tm
Late Pleistocene to early Holocene (~0.02 million years)
6
PRESENT CONDITIONS - LONGTERM CREEP OF LANDSLIDES ONTO VALLEY FLOOR
QTsc
Dutard Creek
PWTP
QTsc
KJb
Tm
KJb

Note: 1) Modified after Vanderhurst, 1981
2) See Figure 13 for explanation of geologic units
3) File: W:\geology\nelson\pwtp\graphic\penbl2.cdr

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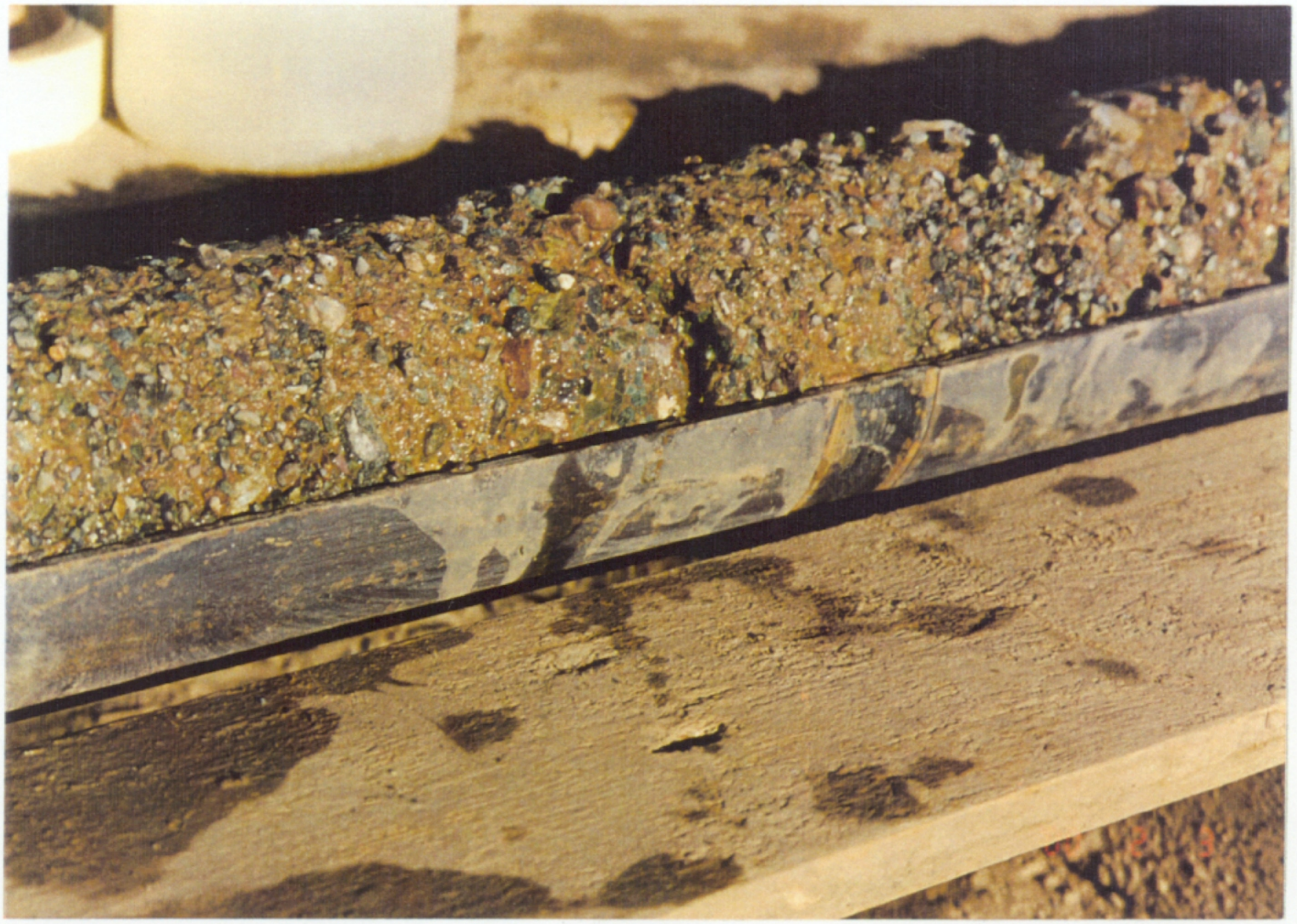
Santa Clara Valley Water District

Schematic Historical Geologic Evolution
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
none

FIGURE
18



Clayey sand with fine gravel in DWR-6 (282.0 to 283.0').

Reference: DWR Memorandum dated 8/7/2000 from Department of Water Resources Project Geology Section to Mike Inamine and Frank Glick, subject "South Bay Aqueduct, Terminal Tank Landslide Investigation, Results of Geological Exploration Program."

file: W:\geology\penitencia creek landslide\figures\fig19.cdr

DATE *1/11/01*	**Santa Clara Valley Water District**	DWR Photograph of Core Material Encountered Beneath Landslide in DWR-6 Penitencia Creek Landslide Penitencia Water Treatement Plant	SCALE none
DESIGN *JLN*			FIGURE 19
DRAWN *JLN*			
CHECKED *RLV*			

4.0 ENGINEERING PROPERTIES OF SLIDE MATERIALS

In the mid-1980s, the District retained R. L. Volpe & Associates (RLVA) to investigate the geotechnical and shear strength characteristics of earth materials near the main shear surface of the Penitencia Creek Landslide. The District provided RLVA with undisturbed samples for laboratory testing from the vicinity of the landslide shear surface during drilling of boreholes to install inclinometer casing and piezometers (Boring No. 8PZ through 14PZ). Some of the collected samples possessed well developed, slickensided shear surfaces, indicative of active landslide movement (RLVA, 1987). However, it is possible that these shear surfaces were auxiliary and not representative of the fully developed principal shear surface. The District collected samples for laboratory testing during the installation of inclinometers I-26 and I-27 (Woodward Clyde, 1997). The District was also provided the laboratory test results obtained by DWR during the installation of three deep inclinometers (DWR-6, DWR-7 and DWR-8) in the vicinity of the DWR Terminal Reservoir installed during the winter/spring of 2000 (DWR, 2000).

Table 2 presents a summary of the geotechnical engineering properties of clay materials from the Penitencia Creek Landslide based on the various sources referenced above. In addition, Table 2 provides a comparison between data from the Penitencia Creek Landslide and geotechnical properties of other Santa Clara Formation claystone samples from a landslide prone region in Saratoga, California (Nelson 1985, 1992). Saratoga is located about 15 miles southwest of Penitencia Creek Landslide. The correlation of the Penitencia Landslide material with that from the Saratoga region is further developed later in this chapter.

4.1 Plasticity of Materials

The Atterburg limits have been widely used in geotechnical engineering since their potential value was first indicated by Karl Terzaghi in the mid 1920's. Terzaghi correctly noted that the results of simplified soil tests, such as the Atterburg limits, depend precisely on the same physical factors, which determine the shear strength and permeability of soils. The Atterburg Limits are based on the concept that a fine-grained soil can exist in any of four states depending on its water content. Thus a soil is solid when dry, and upon the addition of water proceeds through the semisolid, plastic, and finally liquid states. The water content at the boundaries between the adjacent states are termed shrinkage limit, plastic limit, and liquid limit. In spite of their widespread use it was not until the early 1950's that any further serious attempt was made to utilize the limits for more closely identifying the nature of the clay particles present in a soil. A. W. Skempton (Skempton, 1953) then suggested that since the plasticity of the soil was mainly attributable to the clay-size particles present in the soil, the activity ratio (or activity) of the clay is defined by the expression:

$$\text{Activity of clay} = \frac{\text{Plasticity Index}}{\text{Percent clay size } (<2 \text{ microns, } 0.002\text{mm})}$$

Skempton was the first to show that for geologically related clayey soils, an approximately linear relationship existed between the plasticity index and the percent clay sizes, with Activity values ranging from about 0.4 for kaolinite to about 5 for montmorillonite. Further, this term has been very useful for classifying the nature of clay components in different soils.

Table 2. Comparison of Geotechnical Properties of Santa Clara Formation Claystone

Geotechnical	Penitencia Creek Ls*				Saratoga**			
Properties	min	max	average	n	min	max	average	n
Dry Density lb/ft^3	99.7	120.5	111.7	10	82.0	122.0	104.0	70
% Moisture	15	25	19	10	10	37	21	70
Plasticity Index	18	63	40	24	21	68	40	30
Liquid Limit	32	80	58	24	41	82	61	30
% finer than 2 mm	25	55	39	10	18	70	39	30
Activity Ratio	0.73	1.48	1.12	10	0.7	1.57	1.00	30
Residual Friction Angle+	12.1	21.1	16.6	10	6	24.5	10.6	20
Frict. Angle of Slide Surf.	----	----	----		4.6	8	6.3	2
Clay Mineralogy	smectite***				smectite***			30

*Penitencia Creek landslide data from R.L. Volpe & Associates (1987); Woodward-Clyde (1997); DWR (2000)
** Saratoga, California data from Nelson (1985, 1992)

*** x-ray diffraction analysis
+ multiple reversing direct shear and triaxial shear on intact and fabricated samples

n = number of samples/tests
---- = not applicable

To help identify the type of clay present in the Santa Clara Formation and involved in the landslide, an assessment was made of the plasticity characteristics of the material. The plasticity of 24 samples of materials from randomly different locations and depths from the Penitencia Creek Landslide is presented in Figure 20 in the form of the Liquid Limit (LL) versus the Plasticity Index (PI). The LL and PI are part of the Atterburg Limits laboratory test as set forth in the American Society for Testing and Materials (ASTM test method D-4318. The values for both LL and PI are in percent water content based on dry weight. As shown in Figure 20, 5 of the 24 samples classify as clay of low plasticity (CL) in accordance with the Unified Soil Classification System (USCS). The other 19 samples classify as clay of high plasticity (CH) in accordance with USCS.

In reviewing the scatter of the plasticity data presented in Figure 20 it is interesting to note that there is a strong correlation of the data along a trend sub-parallel to the "A" line on the plasticity chart. The "A" line separates clays from silts. Previous research (Seed et al., 1964) has shown that for geologically related soils, the activity of the clay minerals can be estimated from the plasticity chart based on the slope of the best-fit line through LL and PI data points. An increase in the slope of the best-fit line indicates higher activity, while a decrease in the slope indicates a relatively low activity. According to the results presented on Table 2, the activity as determined from PI and percent clay fraction for the Santa Clara Formation is about 1.0 for the 40 samples tested. As shown in Figure 20, the average slope of the LL versus PI data is about 0.87 which, according to previous research (Seed et al., 1964) is representative of an activity of between 2-2.5. Some of the plasticity data, especially the latest Atterberg Limit data (DWR, 2000), suggest that the clay activity could be as high as 4-5. These higher values suggest that activity determined from the plasticity chart may provide more realistic values than activity derived from percent clay size as originally defined by Skempton (1953). It also suggests that the presence of smectite clays may be slightly masked when measuring the percent of clay fraction (minus 0.002 mm) which may tend to lower the computed activity. The influence of the smectite clay on the shear strength of the Santa Clara Formation material is discussed in Section 4.3 in more detail.

4.2 Concepts of Various Shear Strength

Basically, the shear strength of soil is the maximum shear stress (J*f*) acting on a shear slip surface along which failure is occurring. There are three distinct shear strengths, depending on the type of material and the magnitude of strain: peak, critical (or ultimate) and residual.

The concept of residual shear strength is particularly relevant to clay materials. It represents the lowest strength that occurs as a result of very large shear displacements. For sands the residual strength is the essentially the same as the critical state strength. For clays, however, the residual is about ½ the critical state strength. The reason that the shear strength for clay decreases with increasing shear is that, with increasing magnitude of shear, the flat clay particles become aligned parallel to the direction of shear. The residual strength may develop after very large (>2-3 ft) movements have occurred and is not usually pertinent to geotechnical engineering project where generally ground movements, as a result of construction, must be relatively small. However, in an old landslide, such as the Penitencia Creek Landslide, there may have already been very large movements and in such cases the shear strength may already be at the residual value before construction starts. Residual shear strength applies to clays after very large shear displacements when the clay particles have become aligned in well-defined shear zones or slip planes. In the case of the Penitencia Creek Landslide, the shear plane exhibits "slickenside" surfaces, a polished, striated surface caused by one side (the upper side) of the rock mass sliding over another and causing a highly polished slip surface.

Skempton (1964) provided a method for analyzing loss of peak strength in his Fourth Rankine Lecture, where he developed the concept of a Residual Factor (R), which is a relationship between the peak shear strength (sp), average shear strength (save) and residual shear strength (sr). At peak strength the value of R is zero while for fully mobilized residual strength the R value is unity or one. It can be argued that the peak strength is probably mobilized at the outset of progressive failure. Now, as in each case where failure has occurred, the factor of safety must be equal to 1.0 and the actual average shear strength of the clay at the time of failure (save) must be equal to the average shear stress (Jave). Thus by comparing the value of save with the values of sp and sr we can determine whether the strength of the clay involved in the failure was at the peak, at the residual or, perhaps, at some intermediate value. If the concept of progressive failure is valid for overconsolidated clays, one would expect to find that the average strength of these clays decreases with time. Furthermore, it could be argued that failure occurs when the average shear strength has reached a value equal to average shear stress, which is considered to have been acting continuously since the slope had formed. This line of argument also suggests that once the failure surface has been developed for a given slope, its location probably doesn't deviate substantially with time. This is especially true for the Penitencia Creek Landslide where the residual strength is about one-half the average shear strength (see Table 2) as further discussed below.

4.3 Residual Shear Strength of Santa Clara Formation Claystone

RLVA (1987) performed direct shear and triaxial shear strength tests on intact clay samples in an attempt to determine the residual friction angle of the Santa Clara Formation materials. Large strains were obtained during direct shear testing by reversing the strain direction. Residual shear strength data from the direct shear test of clayey material from Penitencia Creek Landslide indicated an apparent minimum residual friction angle of 15.8 degrees (RLVA 1987). Residual shear strength data from the triaxial shear tests indicated an apparent minimum residual friction angle of 12.1 degrees. A careful review of the RLVA tests indicate that even though the tests were carried to large strain, it is most likely that the strains were insufficient to develop residual shear strengths. It is interesting to speculate that the mobilized shear strength along the failure plane can actually be higher than the residual strength depending on water content and other antecedent conditions. The residual shear strength obviously represents a critical or limiting condition.

Nelson (1985, 1992) performed direct shear tests on a claystone sample with a well-developed, intact landslide shear surface from a claystone block to determine residual friction angles. The sample had been hand-carved by a geologist from the sidewall of a 30 in diameter borehole at a depth of about 28 feet. Prior to testing, the direction of shear, indicated by well-developed slickensides on the shear surfaces was aligned parallel to the direction of movement of the direct shear machine. Residual shear strength test results along the intact landslide shear surface from Saratoga, California (Nelson 1985, 1992) indicated that the friction angle depended on the direction of shearing in the direct shear machine. In one direction (arbitrarily defined as the ‘forward’ direction), the residual friction angle was about 4.6 degrees and in the opposite (‘reverse’) direction, the angle was about 8 degrees. Nelson (1992) concluded the following.

> *“The lower residual friction angle measured in the ‘forward’ direction for the intact landslide shear surface is probably due to a preexisting preferred orientation of smectite clay minerals. The lower residual friction angle may be an accurate measurement of the residual shear strength at a point on the landslide surface.”*

As indicated by Table 2, Santa Clara claystone samples from Saratoga, California have geotechnical index properties similar to samples from Penitencia Creek Landslide. However, residual friction angles from the intact landslide shear surface tested by Nelson (1985, 1992) are substantially lower than residual friction angles reported for the Penitencia Creek Landslide by RLVA (1987). According to RLVA (1987) although some of the samples collected from Penitencia Creek Landslide possessed slickensides, actual slide surface material was probably not present in the samples. It is highly probable, therefore, that the principal landslide shear surface may not have been tested by RLVA.

Results of x-ray diffraction analysis of samples from the Penitencia Creek Landslide (RLVA, 1987) and Saratoga area (Nelson 1985, 1992) indicated the clay fraction (% finer than 0.002 mm) of Santa Clara Formation claystone consists predominantly of smectite clay mineral. Smectite is a kind of clay produced when water chemically alters primarily ferromagnesian minerals, calcic feldspars, and volcanic glasses. Smectite clay absorbs water and swells substantially. Each crystal of the mineral is made up of a system of alternating sheets of aluminum-oxygen and silicon-oxygen molecules. Between the sheets the air is charged with an electrostatic force that draws in water, so that the crystal swells up to 8 times its original size. Bentonite, also known as montmorillonite, is made up primarily of the mineral smectite. Bentonites are unique: when dry, bentonite is hard and water tight, and when wet, it has a greasy, slippery feel. It is highly water absorbent. Based on the Atterburg Limits previously discussed, we know that the Santa Clara Formation claystone is not comprised of pure bentonite since bentonite typically has a Liquid Limit of about 300-500 percent and a Plasticity Index of between 250 and 400 percent.

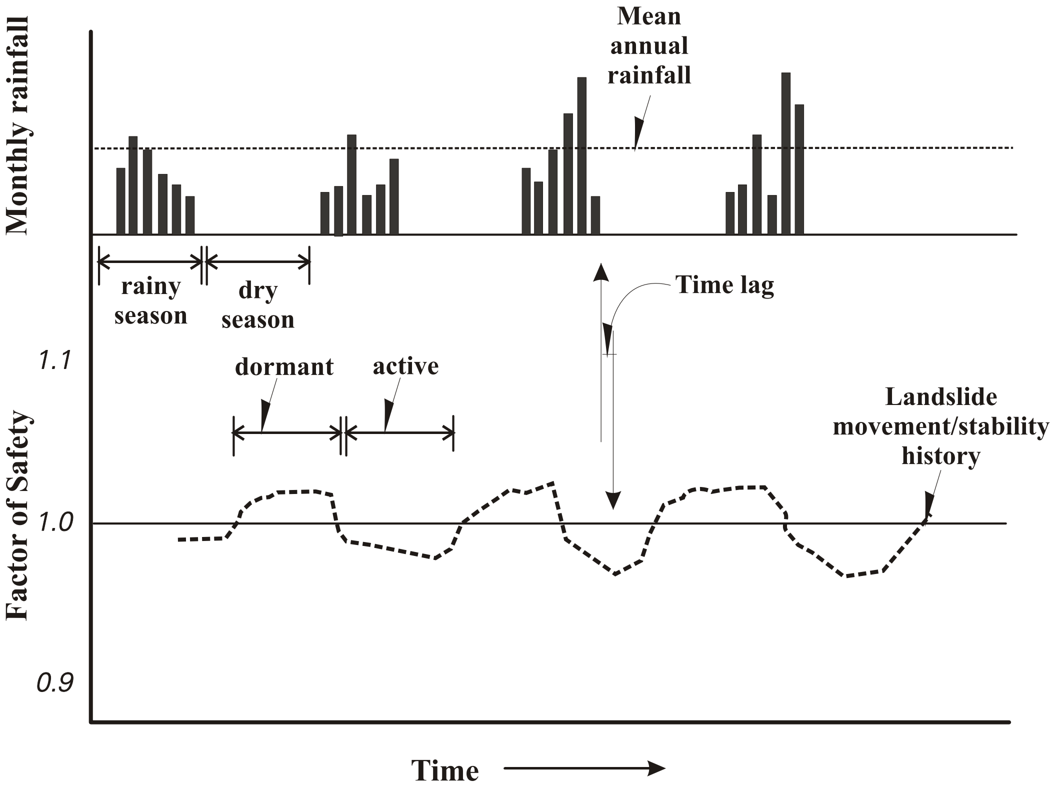
As initially discussed by RLVA (1987), comparison of the results of residual friction angle with the properties of the corresponding plasticity of the clay has been the focus of previous research efforts. For example, Kanji (1974) suggested that the relationship between residual friction angle and plasticity index could be represented by the relationship:

$$N'_{r} = 46.6 / PI_{0.446}$$

Using the relationship by Kanji and the plasticity data presented in Figure 20, the effective friction angle for the Santa Clara formation is computed to range between 12.2° for PI=20 and 7.5° for PI=60. These results compare favorably with the results presented by Nelson (1985, 1992) discussed earlier in this chapter. Research has also shown that residual friction angles less than about 11 degrees indicate the presence of sodium smectite clay minerals (Kenney, 1967).

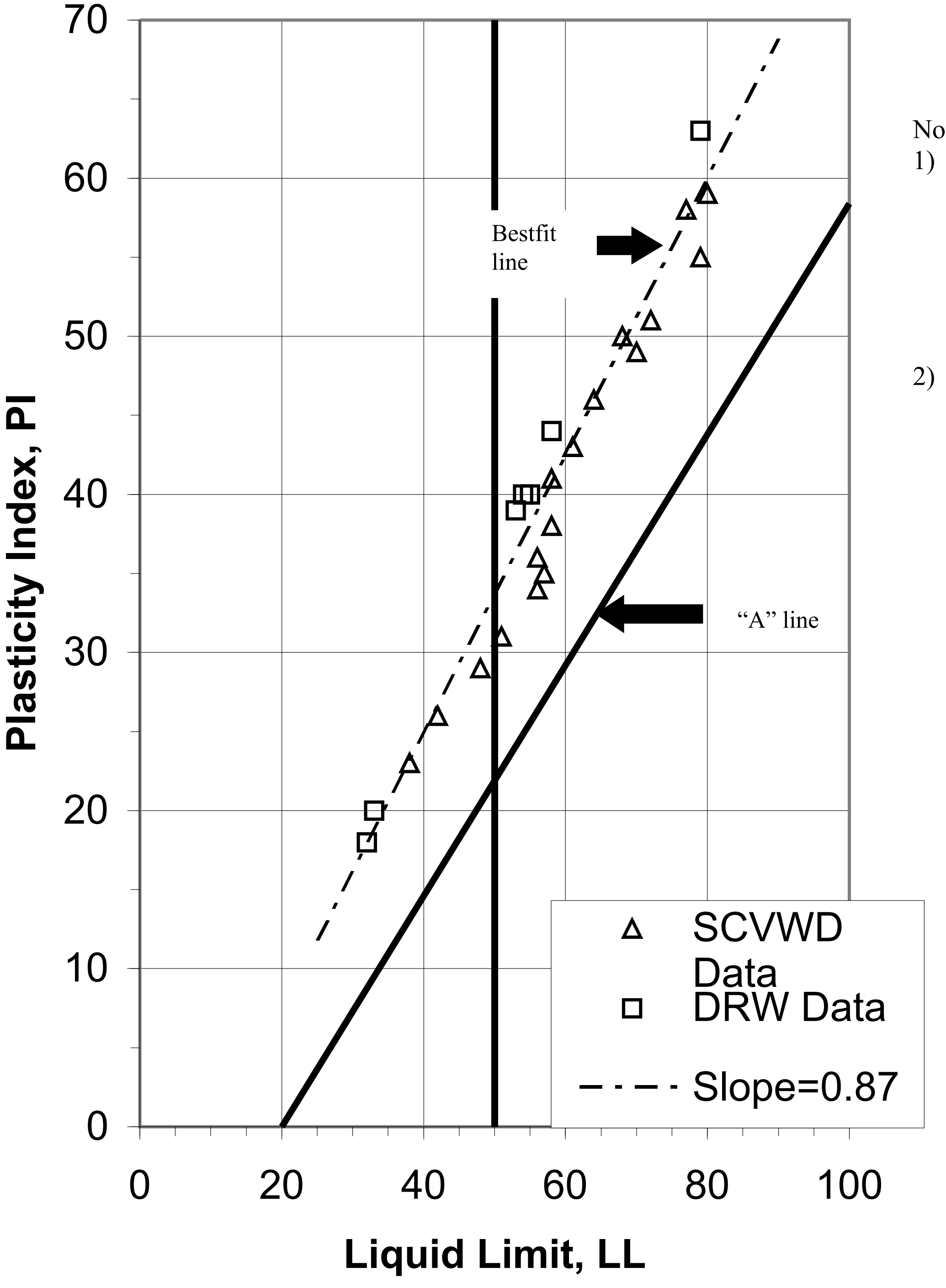
4.4 Mobilized Shear Strength

Due to the complexity of a large landslide, similar to that of the Penitencia Creek Landslide, it is helpful to present a discussion of the concept of mobilized shear strength. For the purpose of this report, mobilized shear strength is defined as that fraction or percentage of the residual shear strength acting or "mobilized" along the actual failure surface in order to resist the combined effects of the cumulative shear stresses. The percentage of mobilized shear strength varies throughout the year (due to rainfall and corresponding pore pressures) and determines whether or not the landslide mass is either stable or unstable. In other words, if the combined summation of shear stress acting along the existing failure plane is less than the residual shear strength, then the landslide mass is stable and movements do not occur. This means that the accumulated induced shear stress is less than the residual shear strength and the computed factor of safety is nominally greater than unity. If, on the other hand, the accumulated shear stress acting along the existing failure plane is greater than the residual shear strength, then the landslide mass is unstable and movements will occur. The magnitude of the incremental movement is controlled by the duration of time that the stress imbalance occurs. The important point to emphasize here in understanding what causes the landslide to move is that the maximum shear strength that can be mobilized along the failure plane is limited to the residual shear strength. It can go no higher. For the Penitencia Creek Landslide, since there are no major changes in surface loading or unloading of the landslide mass, then the mobilized shear strength acting on the failure plane is controlled by the effective stress. Since the total stress acting within the landslide mass remains essentially constant, the effective stress is directly impacted by the variation in pore water pressure acting within the landslide surface. When the pore pressure increases during the rainy season, the effective stress decreases. Conversely, when the pore pressure decreases in the summer months, the effective stress increases. As an aid to visualize how the mobilized shear strength controls the factor of safety and varies at different times throughout the year, a plot of a hypothetical factor of safety during the winter and summer periods is presented in the following figure.



As shown above, the differences in mobilized shear strength between summer and winter may be sufficient to cause the factor of safety to temporarily drop below unity. A relatively minor rise in pore pressure between summer and winter conditions can cause the landslide to either become active and move in the rainy season or become dormant and stop during the dry season. When the landslide is dormant, its factor of safety is nominally greater than unity. When the landslide is active, its factor of safety is nominally less than unity. Also, as shown in the schematic, there is a time lag after the onset rainy and the before the beginning of the dry season before the trend in stability from active to dormant, or visa versa, can physically take place. This has to do with the complex drainage conditions acting within the slide that allow surface water to be converted into or somehow influence the piezometric conditions acting within the landslide. The above scenario does not account for transient loading such as those due to earthquakes. It appears likely that if an earthquake were to occur during the summer or fall months, when the landslide could be dormant, the effects of earthquake-induced displacement would not be as destructive if the earthquake were to occur during the winter or spring season when the landslide is actively creeping. This points out the importance of antecedent rainfall conditions to the potential severity of earthquake-induced landslide movements. Chapter 3 describes in more detail the hypothesized landslide origination. Chapter 7 describes in more detail the hypothesized landslide mechanism.

As discussed in Section 7.4.1.5, we estimate that the Penitencia Creek Landslide has moved as much as 1500-2000 feet out onto the valley floor. It is interesting to speculate on the interaction of the Santa Clara Formation and the underlying older alluvium. We believe that the slide plane is fully contained within the Santa Clara Formation. The friction angle between the contact of Santa Clara Formation and the granular older alluvium is unknown, but may range between 15-20 degrees. If this were the governing shear strength controlling potential movement of the lower portion of the landslide it is speculated that the landslide would have ceased movement long ago. In order for the landslide to continue its relentless, interminable movement out onto the valley floor, the slide plane along which movements are occurring must continue to be confined solely within the Santa Clara Formation with a residual friction angle of between 4 and 8 degrees.



Notes:

1) The average slope of the dashed line equals 0.87, which represents an activity ratio ranging from 2 and 2.5 according to Seed et al. (1964).
2) The measured activity ratio from other laboratory test data ranges 0.73 to 1.48, suggesting a masking affect of the presence of smectite.

DATE
1/10/01
DESIGN
JLN
DRAWN
JLN
CHECKED
RLV

Santa Clara Valley Water District

Plasticity Data of Landslide Materials

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
none
FIGURE
20

5.0 INSTRUMENTATION FOR MONITORING LANDSLIDE CREEP

This chapter presents a discussion of the various instruments and field methods that were used to determine the rate and magnitude of movement of the Penitencia Creek Landslide. A detailed discussion of the actual magnitude, orientation, and computed rates of movement are presented in Chapter 6.

5.1 Inclinometers

5.1.1 System Components

Inclinometers are used to monitor potential horizontal and vertical landslide movement. When using the term inclinometer, the implied system components consist of the following:

- Inclinometer casing is installed in a borehole that passes through suspected zones of movement. Important features include the diameter of the casing, the coupling mechanism, groove precision and straightness, and the strength of the casing. Although some of the earlier casing used at the site was made of aluminum, the majority of casing used is 10 feet long segments that are 3.34-inch diameter and made of high-impact ABS plastic. The casing is fabricated with continuous longitudinal grooves at the 0°, 90°, 180°, and 270° locations around the perimeter of the casing. When installing the casing, it is important to orient the grooves such that one set is normal to the slope contour (assumed direction of landslide movement) in an effort to have one axis that measures the total, or resultant amount of landslide movement.

- An 18-inch long portable inclinometer probe (which consists of an extremely sensitive pendulum) is used to survey the verticality of the casing. The initial survey after installation establishes the initial profile of the casing. After landslide movement has occurred, subsequent surveys reveal changes in the profile of the casing. These surveys are then compared to the initial survey so that the magnitude and direction of movement can be determined. The portable inclinometer probe is the standard device for surveying the casing and is designed to obtain a complete profile because it is drawn from the bottom to the top of the casing. Typically, one reading is taken every 2-foot-length of casing. The depth intervals at which readings are collected do not change throughout the life of the inclinometer. Two sets of readings are taken at the same depth increments along each set of opposing grooves. It is customary to lower the instrument to the bottom of the hole and take readings from the bottom to the top.

- A portable readout or a data logger is used to record the surveys. Although the earlier readings were manually recorded, the more recent readings (about the last 10 years) have been completed with advanced readouts that store readings in solid-state memory, eliminating pencil, paper, and transcription errors, and that enable the transfer of the data to a computer for processing.

- Computer software is used for data reduction and graphing. Inclinometers generate more data than do other types of sensors. A single survey may generate several hundred data points. Over time, tens of thousands of data points are manipulated, reduced, graphed, and archived. Although the earlier data were manipulated by hand, the District has used the SINCO DigiPro software to speed this process.

It should be noted that the 10-ft long casing used for this study are capable of monitoring about 2.5 inches of slide displacement before the degree of curvature becomes so great that the 18-in long inclinometer probe can no longer pass through the disturbed zone. In fast moving slides, inclinometer casing can actually be sheared off. For creeping landslides such as the one at Penitencia Creek Landslide, the distortion due to discrete landslide movement will generally appear at the first joint above the zone of landslide movement. The casing is not sheared off, but rather, the entire column or string of casing above the slide zone moves downhill relative to the string of casing below the slide plane. Only one casing is typically affected and the articulation of the joint coupling that connects one casing to the next has distorted such that the inclinometer probe can no longer pass through the joint.

The movement of the inclinometers installed by the District and DWR are discussed in the following two sections. A detailed description of the location and installation history for each inclinometer is presented Appendix C.

5.1.2 Santa Clara Valley Water District

Since 1972 the District has installed 28 inclinometer borings to monitor landslide movement. Twenty-six of these inclinometers were installed within the boundaries of PWTP and two inclinometers were installed on private property. The District installed inclinometer borings I-1, I-2, I-3, I-4, I-5, and I-6 in 1972. Replacement inclinometers I-3a, I-4a, I-5a, and I-6a were installed in 1975 and 1978. Inclinometers I-10 through I-15 were installed in 1985. Inclinometers I-16 through I-19 were installed in 1986. Inclinometers I-20 through I-25 were installed in 1991. Most recently, inclinometer borings I-26 and I-27 were installed in 1997. Inclinometers I-18 and I-19 situated directly north of the PWTP are inclinometer borings installed on private property. Note that the sequence of inclinometer numbers does not include I-7 through I-9.

In addition, the District has been monitoring inclinometer boring H-4 on a residential parcel situated upslope and northeast of the PWTP on Perie Lane (Figure 3). This inclinometer boring was installed by Frank Lee and Associates in September 1992 and is still operational.

5.1.3 California Department of Water Resources (DWR)

From 1973 to 1984 DWR installed 10 inclinometer borings in the vicinity of the Terminal Reservoir. These consisted of 5 initial inclinometer borings and 5 replacement borings. In the winter and spring 2000 DWR installed 3 new inclinometer borings designated as DWR-6, DWR-7, and DWR-8 upslope of the Terminal Reservoir to monitor landslide movement of the shallow Terminal Tank Landslide and the deeper main slide surface (Figure 3). Inclinometer borings DWR-6 and DWR-8 were drilled to a depth of about 300 feet, while DWR-7 was drilled to a depth of about 100 feet. DWR is sharing their inclinometer data with the District. These data will be analyzed and incorporated into future surveillance reports.

5.1.4 Methodology of Analyzing Inclinometer Data

Lateral movements of the landslide were measured at discrete locations with each casing using an inclinometer probe manufactured by Slope Indicator. District surveyors who were trained in the use of the inclinometer take quarterly field measurements with the inclinometer probe. As mentioned previously, the probe is a very sensitive electronic pendulum and measures tilt of the casing at discrete intervals that remain constant for each inclinometer casing. By comparing subsequent changes in readings, the magnitude and orientation of the movement can be calculated as further discussed below.

As mentioned earlier in this chapter, inclinometer casing is manufactured with two sets of perpendicular grooves, arbitrarily designated as either the "A-axis" (e.g., 0° and 180°) or the "B-axis" (e.g., 90° and 270°) that run the length of the casing. The A+ direction is defined as the orientation of one end of the "A-axis" or primary axis, which is usually positioned during installation to point in the general direction of expected landslide movement. The secondary axis is known as the "B-axis." The orientations of the A+ direction for all the site inclinometer borings are summarized on Table C-1. Because the A+ direction will usually not coincide exactly with the direction of landsliding, the cumulative data plots will not show the maximum, or resultant, displacement. The orientation of the cumulative displacement plots is adjusted using DigiPro software to determine the resultant displacement. The resultant deflection orientation is determined by directing DigiPro software to "rotate" the inclinometer plot clockwise (or counter clockwise), until the maximum apparent deflection of the A+ direction plot is achieved. The actual method used is based on conventional geometric relationships. For example, the magnitude of movement is computed using the Pythagorean relationship, which can be expressed as follows:

$$C = (\Delta A^2 + \Delta B^2)^{1/2}$$

where;

ΔA = differential movement of the A axis

ΔB = differential movement of the B axis

C = resultant total movement of the slide plane

The orientation of movement between successive movements can be determined using geometric relationships of the incremental movement of the A and B axis. Taking care to note the arithmetic sign of incremental movements, the corresponding angle of slide movement is determined using the following relationship:

$$\tan(\theta) = \Delta B / \Delta A$$

Where;

$\tan(\theta)$ = tangent of angle of inferred orientation of slide movement

5.1.5 Accuracy of Inclinometer Instrument

The Slope Indicator Company states that the system accuracy of the inclinometer is ±0.3 inches per 100 feet (2.5 x 10-4) of casing, assuming proper reading techniques are used. The specification was derived empirically from a large number of datasets and includes both random and systematic error for 50 reading increments and a 2-foot inclinometer probe similar to the one used at PWTP. For a single reading increment, random errors were found to be 0.007 inches and systematic errors were found to be 0.005 inches. A random error accumulates with the square root of the number of reading increments. The total error for a 100-foot traverse (50 reading increments) can be calculated as:

Total Error = Random Error + Systematic Error
= 0.007 * sqrt (50) + 0.005 * 50
= 0.05 + 0.250
= 0.300 inch

Systematic errors can be corrected, but random errors cannot. Thus the limit for precision for 50 reading increments is ±0.05 inch, and the limit of precision for a single interval (such as a slide plane) is about ±0.01 inch. As stated previously, the inclinometer casing used at the Penitencia Creek Landslide have sustained between 2-3 inches of slide movement before becoming unmeasurable.

5.2 Benchmark Distance Shortening Surveys

The California Department of Water Resources (DWR) has surveyed the distance between numerous points on the Penitencia Creek Landslide relative to assumed stable benchmarks off the landslide since 1971. A plan view of the surface monitoring points is presented in Figure 5. The assumed stable monument points, referred to as District reference point "School" and DWR reference point "Geod" are located about 3,000 feet off the toe of the slide on Piedmont Road. The two reference points are located southwest of the landslide and in line with the principal downslope component of landslide movement. The surveys were performed using electronic distance measuring equipment (EDM). The distances measured, hereafter termed "EDM lines" range from about 3,280 feet to 5,905 feet. As the landslide continues to move, the survey results are used to calculate both the magnitude and rate of relative shortening of the EDM lines. It should be noted that the current standard error for EDM equipment is about 0.01 foot ± 3 parts per million. Thus, the standard error for an EDM survey line of 5,000 feet is 0.01 foot ± 0.015 foot (i.e., [5,000/1,000,000]*3), or a repeatability of –0.005 foot and +0.025 foot between readings.

5.3 Slide Monitoring Following Recent Earthquakes

DWR performed surveys of the EDM lines within days prior to and immediately after the Morgan Hill earthquake (Tepel 1985). Thus, the calculated shortening of the EDM lines due to the earthquake is fairly well constrained. DWR had completed horizontal location surveying on April 19, 1984, five days before the earthquake. Because the surveyors were still working on levels in the area on the day of the earthquake (April 24), they were able to re-measure the distance from their bench mark to some of the points located on April 19 (Tepel 1985). These results are presented in Section 6.4.

5.4 Pipe Joint Closure

Three water delivery pipelines run eastward from the floor of Santa Clara Valley over the northwestern toe of the landslide and end at DWR's Terminal Reservoir and the Penitencia Water Treatment Plant. The largest, installed in 1964, is a 72-inch diameter pipe known as the South Bay Aqueduct (SBA) that is owned and maintained by DWR. The other two pipes, installed in 1973, are owned by the District and include a 66-inch diameter pipe known as the Penitencia Force Main and a 60-inch diameter pipe known as the Penitencia Delivery Main.

Landslide movement caused compression failure of the three pipelines at the toe in 1983. The District constructed a reinforced concrete encasement as a temporary fix to get the three pipelines back in service. In October 1984, the District installed Dressor Couplings in combination with triple arch bellows on each pipeline as a permanent fix. The combination fix allowed for a maximum allowable compression of ± 3.5 inches. In March 1993, the bellows were replaced as the original ones had reached their maximum limit. In addition the original concrete vault had experienced extensive failure and was also replaced. A new concrete vault (referred to herein as the "Penvault") was constructed to provide access to and monitoring of the pipe couplings. Both the Dressor couplings and the collapsible bellows were fitted with extensometers (hereafter referred to as "jointmeters") to manually monitor both the magnitude and rate of measured pipe shortening. No extensometers were installed on the 72-inch diameter SBA pipe.

5.4.1 Dressor Couplings

The District monitored movement at the toe of the landslide by measuring closure of the Dressor pipe couplings from early 1985 through March 1993. Closure was measured at approximately monthly intervals by manually taking micrometer measurements across three points of the Dressor coupling on each pipe (typically at the 12:00, 4:00, and 8:00 clock positions, looking upstream (i.e. towards the plant).

5.4.2 Collapsible Bellows

In August 1995 the District installed four vibrating wire jointmeters to electronically measure closure of the collapsible bellows. Two jointmeters were installed across the 66-inch pipe and two jointmeters were installed across the 60-inch pipe. As mentioned above, no jointmeters were installed across the 72-inch SBA pipe. The jointmeter instruments were connected to a Campbell Scientific recording datalogger unit and a modem that are mounted on a wall of the Penvault. Joint closure data are automatically collected by a Campbell Scientific datalogger that daily interrogates and collects data from each jointmeter. District staff retrieves the data using an office computer that can be connected to the modem line.

5.5 Time Domain Reflectometry

Time domain reflectometry (TDR) is a remote sensing technique developed by the communications industry to detect and locate breaks in their buried cables. TDR is analogous to radar in that an electrical pulse or signal is sent through an aluminum shield around the cable, which is then reflected back to the TDR recording instrument. When the pulse encounters a break or deformation in the cable, the pulse is reflected and recorded as a "spike" on the characteristic signature of the cable when measured by a TDR cable readout box. The characteristic signature of a particular cable is determined by electronically sending an initial pulse through the cable after it is first installed prior to any disturbance (bending) of the cable.

TDR is gaining wide acceptance for use in landslide monitoring to determine the depth to a failure surface by installing a cable through a landslide mass. Movement of the landslide along discrete zones will deform and potentially break the cable. Subsequent monitoring of the TDR cable will indicate the presence of a "spike" at the depth(s) where landsliding (discrete movement) has occurred. The advantages of TDR over inclinometer borings include 1) much lower cost to install, 2) rapid monitoring with a readout box, 3) easier installation in difficult terrain, 4) ability to automate readings, and 5) ability to alarm landslide movement. A major disadvantage of TDR is the inability to monitor either the magnitude or rate of landslide movement.

With the cooperation of DWR, the District directed installation of TDR cable in new inclinometer borings DWR-6, DWR-7, and DWR-8 situated adjacent to and upslope of the Terminal Reservoir. In spring and early summer 2000, the District and DWR also installed and grouted TDR cable in inclinometer I-20, which had been abandoned in 1995 since the inclinometer probe could no longer be lowered past the failure plane of the landslide. This was possible since the TDR cable, which is about 1/2 inch in diameter and quite rigid, could be inserted past the location of the slide plane, even though the inclinometer instrument could not.

6.0 ANALYSES AND RESULTS

This chapter presents the calculated rates of movement of the Penitencia Creek Landslide that were determined by analyzing data from the following surveillance instruments or observations; deflection of inclinometer casing, changes in GPS survey monument locations at the PWTP, pipe joint closure near the toe of the slide, and shortening of Electronic Distance Measuring (EDM) survey lines obtained by DWR. Some of the data covers the 29-year period from late 1971 through mid-2000. As mentioned in Chapter 5, the District recently installed Time Domain Reflectometry (TDR) cable in several inclinometer borings at the site to begin collecting a new type of landslide monitoring data. Unfortunately, TDR data collected so far has not registered slide movement. Such data will continue to be collected and reported in future surveillance reports.

The following sections explain the methodologies used to analyze the various landslide movement data. Wherever appropriate, the data plots also show cumulative yearly rainfall in order for the reader to correlate landslide movement with rainfall.

6.1 Landslide Creep Rates Based on Deflection in Inclinometer Casing

6.1.1 Methodology of Analyzing Inclinometer Data

Raw data from the inclinometer probe readouts are converted to lateral movement using Digipro software developed by Slope Indicator. The lateral movement data are typically plotted as cumulative displacement plots, which are used to readily identify shear displacement. As shown on cumulative displacement plots in Appendix C, lateral deflection of the inclinometer casing due to landslide movement is evident as a distinct nearly horizontal offset of the predominantly vertically plotted data. The offset is usually at a particular depth (shown by the vertical scale) that identifies a shear plane or at several depths if more than one shear plane is present. Because the inclinometer casing is deflected as the landslide progressively moves along a discrete shear plane, data plots based on successive field measurements will show successively greater displacement on the cumulative displacement plots. Consequently, to determine the amount of landslide movement, it is necessary to measure the length of offset on a data plot and convert it to inches based on the horizontal scale of the cumulative displacement plot. Rate of movement is calculated by dividing a particular offset distance by the total time of monitoring for a particular inclinometer boring.

6.1.2 Inclinometer Data Results

PWTP inclinometer boring locations in plan view are shown on Figure 3 and their locations in cross section view are shown in Figures 6 through Figure 10. Pertinent inclinometer boring data are summarized in Table C-1 of Appendix C. Displacements along the primary shear surface (in inches) versus time are shown on Figure 21. Displacements along secondary creep zones (in inches) versus time are shown on Figure 22. A plan view of displacement vectors for the primary shear surface is shown on Figure 23. A plan view of displacement vectors for secondary creep zones are shown on Figure 24. The vector size and reported rate are based on cumulative displacement averaged over the time period of monitoring for a particular inclinometer boring. The vector arrow shows direction of relative movement.

For our analysis of vector displacement rates, we did not include accelerated movement due to the Loma Prieta earthquake (inclinometers I-16, I-17, and I-18) or due to unusually wet winters of 1993 and 1995 (I-20). The effect of the earthquake and the wet winters are evident as distinct near-vertical "jumps" in the data plots as evident on Figures 21 & 22.

Based on inclinometer deflection data, from about 1986 to the present, the average landslide movement along the primary shear surface ranged from about 0.13 inches per year (in/yr.) to 0.35 in/yr. During periods of above average rainfall, accelerated landslide movement of between 0.3 inch and 1.1 inches has been measured within a period of less than 6 months. It is interesting to note, however, that the accelerated rates of landslide movement during wet periods appear to quickly return to the lower rates in effect prior to the increased rainfall. Movement along secondary creep zones for the same time period ranged from 0.01 in/yr. to 0.06 in/yr.

6.1.3 Inclinometer Casing Deflection due to the Loma Prieta Earthquake

Deflection of inclinometer casing along the main landslide shear surface due to the Loma Prieta earthquake are evident in Figure 21 as near vertical jumps in the displacement curves. Deflection of inclinometer casing along a secondary creep zone due to the Loma Prieta earthquake is evident in Figure 22. Displacement due to the Loma Prieta earthquake along the main landslide shear surface ranged from 0.40 in to 0.70 in and displacement along the secondary creep zone ranged from 0.10 in to 0.40 in.

6.2 Landslide Creep Rates Based on Changes in GPS Monument Survey Locations

6.2.1 Methodology of Analyzing GPS Monument Survey Data

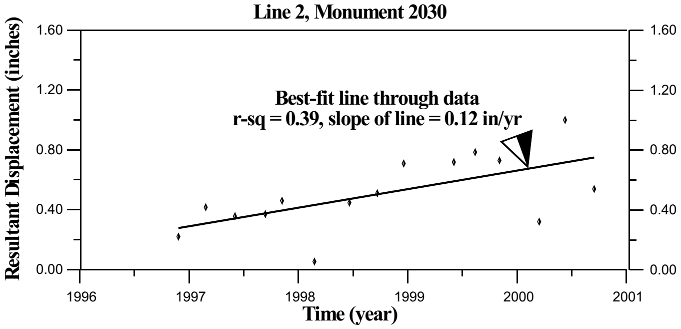
Since 1991, District surveyors have used GPS survey methods to survey locations of four monuments located near the four corners of the water treatment plant process basins (Figure 5). These monuments, referred to as monuments 1, 4, 10 and 13, respectively, consist of brass plate set in concrete. It should be noted that these monuments have been destroyed during the current upgrade work at the plant. New monuments will be established on the upgraded plant and will be surveyed using GPS survey methods in the future.

Since 1996, District surveyors have used GPS to also survey monuments along 6 survey lines established at various locations on the landslide. Each survey line consists of multiple "shiner and nail" monuments positioned along roughly linear street alignments at the locations shown on Figure 5. These monument survey lines were strategically located to monitor landslide movement across suspected boundaries of the landslide, such as the toe or head, or within interior regions of the landslide mass. The monument lines are surveyed quarterly.

The GPS survey data consists of changes in X and Y positions (eastings and northings, or latitude and longitude) relative to an initial survey date. For analysis purposes, the survey data are converted into a resultant displacement vector of each monument. For each vector a displacement rate in inches per year (in/yr.), and direction of movement, are computed. To determine which monuments are exhibiting displacement due to actual landslide movement (as opposed to those monuments whose apparent motion is due to inherent survey errors) the data were analyzed using conventional statistical methods. The statistical analysis produces a resulting best fit line and regression (R) data. The regression data is a measure of the degree of correlation of the best-fit line, with 1.0 indicative of a direct one-to-one correlation. As discussed below in more detail, The regression data were used to determine both the direction of movement and the rate of movement.

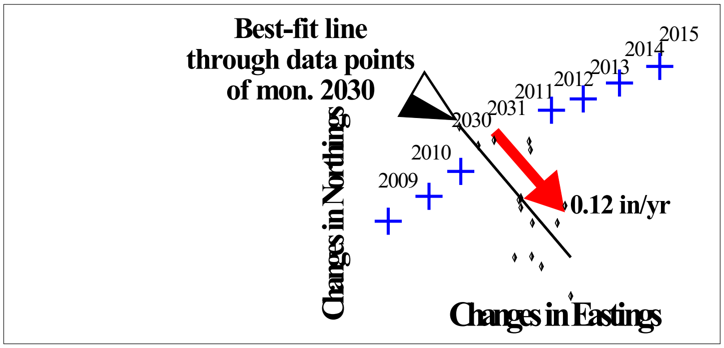
Those monument data with relatively high correlation values were selected as indicative of monuments that have experienced actual landslide movement. Furthermore, it is anticipated that those lines with high correlation values will continue to show good correlation in the future. Conversely, those monuments that are either not located on the landslide or not experiencing landslide movement will continue to show poor correlation with time.

The resultant magnitude of displacement of a particular monument for one quarter (one data point) was calculated by taking the square root of the change in latitude (lat.) squared plus the change in longitude (long.) squared. The resultant displacement rate for the entire time period of survey data collection for a particular monument was determined by plotting resultant displacement versus time and finding the slope of the best-fit line passing through the data. The slope of the best-fit line was chosen as the rate of displacement of a given monument, and thus the rate of landslide creep at that point. Based on our experience, best-fit lines with an R-squared regression value of between 0.1 and 0.2 or greater are considered as being a relatively good correlation and thus indicative of actual landslide movement. A typical regression analysis for survey monument 2030 on Line 2 is shown below.



To determine direction of movement we plotted changes in the GPS latitude (lat.) and longitude (long.) location data. Many of these plan view graphs showed strong linear data alignments in the direction of anticipated landslide movement. Next, we plotted best-fit lines through each monument data plot and determined R-squared values. As stated previously, best-fit lines with R-squared values of 0.1 to 0.2 or greater are considered to represent relatively good correlation and thus indicative of actual landslide movement.

Finally, we drew an arrow parallel to the direction of the best-fit line to indicate direction of movement as shown below.



6.2.2 Monument Survey Results

A plan view plot of the resultant vectors and corresponding rates of displacement for the four treatment plant monuments from 1991 through 2000 is shown on Figure 25. A plan view plot of the resultant vectors and corresponding rates of displacement for the six monument line surveys from 1996 through 2000 is shown on Figure 26. A composite plan view plot showing resultant vectors from all inclinometer borings and monuments is shown on Figure 27.

For the four treatment plant monuments the resultant vector directions ranged from 225 degrees azimuth (da) to 240 da (southwesterly) and the rates of displacement ranged from 0.27 in/yr. to 0.28 in/yr. For this report all azimuth readings are reported to the nearest 5 degrees.

The following paragraphs summarize GPS monument survey results from the six line surveys. A plan view of the line surveys is shown in Figure 5. Monument line 1 is located on the southeastern portion of the landslide mass on upper El Grande Drive. Monument line 2 is located near the southeastern toe of the landslide on lower El Grande Drive. Monument line 3 is located near the southeastern toe on Noble Avenue. Monument line 4 is situated north of the central toe of the landslide mass on Whitman Avenue, immediately north of the Penvault. Monument line 5 is situated near the northwestern toe on lower Suncrest Avenue near its intersection with Burgundy Drive. It should be noted that monuments of line 5 were paved over and are no longer accessible for surveying. New monument will be established in the near future. Monument line 6 is situated near the landslide headscarp on upper Suncrest Avenue.

All five survey points of monument line 1 appear to be moving due to landslide creep (Figure 28). Monument line 1 resultant vector directions ranged from 200 da to 220 da (southwesterly) and the displacement rates ranged from 0.07 in/yr. to 0.58 in/yr. Of all the instrumentation data analyzed for landslide movement, the southernmost monuments 1049, 1313, and 1050 exhibit the greatest displacement rates of 0.52 in/yr., 0.58 in/yr., and 0.56 in/yr., respectively. Consequently, we have mapped this area as being underlain by a relatively more active smaller subsidiary landslide mass.

Six of the 20 survey line 2 monuments appear to be moving due to landslide creep (Figure 29). Monument line 2 resultant vector directions ranged from 120 da to 140 da (southeasterly) and the displacement rates ranged from 0.05 in/yr. to 0.12 in/yr. On Figure 29 we mapped the toe of the landslide mass as a thick dashed line passing between the group of monuments (2009, 2010, 2030, 2031, 2011, and 2013) that appear to be actively moving and those that do not.

Three of the 9 survey line 3 monuments appear to be moving due to landslide creep (Figure 30). Monument line 3 resultant vector directions ranged from 95 da to 120 da (east-southeasterly) and the displacement rates ranged from 0.08 in/yr. to 0.13 in/yr. On Figure 30 we mapped the landslide toe as a thick dashed line passing between the three northeastern-most monuments (3006, 3007, and 3301) that appear to be moving and the group of monuments to the southwest that do not appear to be moving.

Seven of the 11 survey line 4 monuments appear to be moving due to landslide creep (Figure 31). Monument line 4 resultant vector directions ranged from 210 da to 220 da (southwesterly) and the displacement rates ranged from 0.08 in/yr. to 0.33 in/yr. It should be noted that the seven monuments that have shown movement have exhibited an average rate of displacement of 0.24 in/yr. This agrees fairly well with pipe joint closure data (see below). The rate of closure data ranges from 0.17 in/yr. to 0.37 in/yr. with an overall average of 0.29 in/yr. On Figure 31 we mapped the landslide toe as a thick dashed line passing between the 7 northeastern-most actively creeping monuments (4404, 4405, 4040, 4006, 4007, 4409, and 4410) and the remaining group to the southwest that do not appear to be moving.

Two of the 7 survey line 5 monuments appear to be moving due to landslide creep (Figure 32). Monument line 5 resultant vector directions ranged from 210 da to 230 da (southwesterly) and the displacement rates were 0.25 in/yr. and 0.31 in/yr. On Figure 32 we mapped the landslide toe as a thick dashed line passing between the two northeastern-most monuments (5032 and 5004) that appear to be moving and the group to the southwest that do not appear to be moving.

All eight of survey line 6 monuments appear to be moving due to landslide creep (Figure 33). Monument line 6 resultant vector directions ranged from 220 da to 240 da (southwesterly) and the displacement rates ranged from 0.03 in/yr. to 27 in/yr.

6.3 Penvault Pipe Couplings Closure Rates

Since 1985 the District has collected pipe joint closure data at the Penvault. As previously mentioned, between 1985 and 1993 the District manually recorded closure data in the form of micrometer readings across the triple arch bellows. From 1995 to the present the District has downloaded electronic closure data from jointmeters positioned across collapsible bellows. Figure 34 is a plot of the manual closure data versus time and electronic closure data versus time for the 60-inch Penitencia Main (Figure 34A) and the 66-inch Penitencia Force Main (Figure 34B).

For our analysis we plotted pipe closure in inches versus time. The general downward trend of the data for each monitoring point indicates progressive closure of the joints due to landslide creep at the toe of the landslide. As previously mentioned, the superimposed valleys and peaks are due to temperature changes. To analyze the rate of closure, and thus determine the rate of landslide creep at the toe, we plotted best-fit lines through the plotted data (Figure 34A and 34B). The slope of the best-fit line was chosen as the rate of average pipe joint closure (landslide creep).

As indicated in Figures 34A & 34B, the pipe joints have experienced steady closure due to compression at the toe of the landslide. These plots show overall general downward trends with a series of distinct superimposed peaks occurring in mid-winter and valleys occurring in mid-summer. These sine-wave forms of movement reflect seasonal temperature fluctuations of the water as it is transported through the pipelines. Thermal expansion of the pipe joint occurs as warmer water passes through the pipes, resulting in a downward trend (valley) whereas thermal contraction of the joint occurs as colder water passes through the pipes, resulting in an upward trend (peak).

Data plots and best-fit lines through the pipe joint closure data are shown on Figure 34. The following are estimated rates of landslide creep in in/yr. at the toe of the landslide in the vicinity of the Penvault based on the slope of best-fit lines through the pipe closure data. The manually recorded closure data measured on the 60-inch Penitencia Main from 1985 through 1993 provided landslide creep rates of 0.29 in/yr. and 0.30 in/yr. The manually recorded data from the 66-inch Penitencia Force Main provided closure rates of 0.33 in/yr. and 0.37 in/yr. The electronically recorded closure data measured on the 60-inch pipeline from 1995 to the present provided landslide creep rates of 0.17 in/yr. and 0.20 in/yr. The electronically recorded data from the 66-inch pipeline provided a closure rate of 0.32 in/yr.

6.4 EDM Line Shortening Rates

6.4.1 Methodology of Analyzing EDM Line Shortening Data

Since 1972, the California Department of Water Resources (DWR) has performed surveys to monitor landslide creep of the Penitencia Creek Landslide using Electronic Distance Measuring equipment (EDM). DWR has established and maintained over a dozen monuments at various locations on the Penitencia Creek Landslide, especially near the shallow 3-acre landslide immediately uphill from the Terminal Reservoir. The location of seven representative EDM monuments (out of a total of 13) is shown in Figure 5. Landslide creep is measured by determining the shortening of the survey line measured from each monument to a stable point DWR designated as "Geod," which is located about 2700 feet southwest of the toe of the landslide on Piedmont Road (Figure 5). Since only one EDM station is analyzed, just the magnitude of line shortening (i.e., not magnitude and orientation) is computed. For our analysis, we plotted the magnitude and rate of shortening in inches of three EDM lines versus time. The EDM lines established for monuments A, C and D-1 were used because the EDM data for these monuments covered the longest period, from about 1972 to 1997 (~25 years), and the fact that these three monuments are located in close proximity to the PWTP. The shortening for each monument as a function of date is summarized on Table 3. The individual data for the three survey points presented on Table 3 are plotted as a function of time in Figure 35. The results of our analysis of the magnitude and rate of landslide creep are presented in the following section.

6.4.2 EDM Line Shortening Results

As shown in Figure 35, the EDM surveys have monitored a total shortening of the landslide surface of about 11 to 12 inches during the 28+ years of survey. The line shortening appears to be reasonably consistent for the three different monuments. In a gross sense, this magnitude of shortening amounts to a shortening rate of about 0.5 in/yr. However, a review of Figure 35 suggests that the rate of the actual line shortening has not been consistent during this period. Initially, we attempted to analyze the rate of shortening using a statistically determined best line through the individual data. It was obvious, however, that the subsequently computed movement rate was impacted by the earthquake-induced movement of the April 24, 1984 Morgan Hill. Including the effects of the earthquake imparted a significant bias to the calculation of rate of landslide movement. It was decided therefore to compute a simple average for the rate of shortening using the following approach. Two separate movement rates were determined before and after the Morgan Hill event for each survey point as shown in Figure 36. The rate of movement occurring prior to the Morgan Hill event was computed as follows: the rate of movement at any particular time prior to the earthquake is equal to the total movement up to that time divided by the total elapsed time of survey. The rate of movement occurring after the Morgan Hill event was computed as the total movement divided by the total time of survey since the date of the earthquake for the particular monument.

As shown in Figure 36, the average rate of movement (shortening) of survey point A was between 0.3 and 0.4 in/yr. prior to the Morgan Hill event. The sudden spike in movement rate of about 1.8 in/yr. is for the first reading immediately after the earthquake. In subsequent readings following the earthquake, it is apparent that the trend in movement rate is decreasing from the unusually high value, but it does not immediately return to its pre-earthquake rates. In fact, during an approximate twelve-year period following the earthquake from October 1984 through early 1997, the movement rates for monument A have been steadily decreasing but are still higher than the rate prior to the earthquake. The movement rates for survey points C and D-1 are not as clear as that for survey point A, although they both appear to be slightly slower. For example, the computed average movement rates on April 19, 1984 (one week before the earthquake) for survey points A, C, and D-1 were 0.44, 0.35, and 0.39 in/yr., respectively. These rates include total landslide movement from the beginning of the EDM surveys (April 15, 1972 for point A; October 15, 1974 for points C and D-1) to the week before the 1984 earthquake. A similar trend in decreasing movement rates for points C and D-1 can also be seen in Figure 36. The significant increase in rate shown for point A immediately following the earthquake is not seen for points C and D-1, probably because for some unknown reason points C and D-1 were not read following the earthquake until a year later in April 1985. For the last EDM reading taken January 10, 1997

Table 3 - Distance Shortening of Selected Monuments as Measured by EDM

Date	Mon A	Mon C	Mon D-1		Date	Mon A	Mon C	Mon D-1
5/15/72	0.00	-	-		4/24/84	5.55	4.80	4.92
5/15/73	0.35	-	-		10/18/84	6.46	-	-
8/15/73	0.28	-	-		11/14/84	6.46	-	-
5/15/74	0.24	-	-		12/4/84	6.65	-	-
10/15/74	0.98	0.00	0.00		4/15/85	6.54	5.43	5.47
1/15/75	0.94	0.00	0.00		9/26/85	6.26	5.75	5.55
6/15/75	1.02	0.00	0.00		12/6/85	6.89	5.98	5.83
1/15/77	1.73	0.12	0.71		2/20/86	7.17	5.98	6.06
1/15/78	1.61	0.31	0.63		3/3/86	7.32	-	6.30
2/15/78	1.89	0.39	0.83		4/2/86	7.95	6.42	-
3/15/78	1.97	0.39	0.94		1/11/88	8.54	7.56	6.81
10/15/78	2.56	1.02	2.13		4/22/88	8.58	7.52	7.05
11/15/78	2.56	1.10	2.32		6/20/88	8.70	7.68	7.72
1/3/79	2.52	1.02	1.89		7/8/88	8.58	7.83	7.32
2/28/79	2.60	-	-		1/24/89	8.82	7.95	7.83
8/9/79	2.99	-	-		8/23/89	-	-	7.99
1/28/80	2.60	-	-		10/18/89	-	8.70	8.35
3/10/80	2.95	1.93	2.28		5/8/91	10.08	9.17	8.74
4/15/81	2.99	1.85	2.52		2/18/92	-	9.53	8.94
5/15/82	3.46	2.36	2.87		4/20/92	10.71	9.53	9.13
12/23/82	4.21	2.91	3.50		1/20/93	10.98	9.69	-
1/31/83	3.94	2.76	3.27		6/9/93	11.06	9.84	-
2/24/83	4.21	3.19	3.58		1/17/95	11.26	-	10.00
3/22/83	4.80	3.19	4.13		3/17/95	11.42	-	10.08
5/6/83	4.76	3.58	4.09		1/19/96	-	-	10.63
9/14/83	4.72	3.54	4.53		3/1/96	11.93	10.35	10.43
12/20/83	4.88	3.66	4.57		1/10/97	11.93	10.91	10.79
4/19/84	5.20	4.17	4.61					

Notes:

1. The results presented hereon represent Electronic Distance Measuring (EDM) of several monuments located at the Penitencia Creek Landslide.
2. The monument locations are shown in plan view in Figure 5.
3. The EDM shortening results are presented in Figures 35 & 36.
4. The Morgan Hill (M=6.2) earthquake occurred April 24, 1984 and is estimated to have produced a peak ground acceleration of about 0.18 g at the site.
5. The Loma Prieta (M=6.9) earthquake occurred October 17, 1989 and is estimated to have produced a peak ground acceleration of about 0.10 g at the site.

The computed average movement rates since the 1984 Morgan Hill earthquake for survey points A, C, and D-1 were 0.50, 0.48, and 0.46 in/yr., respectively. In conclusion, it appears that the average movement rate of the landslide mass based on EDM survey methods is between about 0.35 and 0.5 in/yr. It also appears that significantly higher rates of landslide movement can exist for a protracted period of time following a moderate earthquake. The higher rate of landslide movement as a result of the earthquake indicates that seismic shaking somehow weakens the slide plane mechanism described in Section 4.4. This protracted strength loss is perhaps due to an as yet undefined rheological or thixotropic component to the slide mechanism. It is analogous to, although in the reverse direction, of the thixotropic regain of strength in Normally Consolidated (NC) clays. For example, it is known that if NC clay is completely sheared to its remolded shear strength, and then allowed to sit, it will begin to regain a portion of its peak shear strength with time.

6.4.3 EDM Line Shortening Due to Earthquakes

The EDM data presented on Table 3 were also reviewed to assess the magnitude of earthquake-induced landslide displacement for two recent earthquakes; namely, the Morgan Hill (M=6.2) earthquake of April 24, 1984, and the Loma Prieta (M=6.9) earthquake of October 17, 1989. Shortening of the EDM lines due to the Morgan Hill earthquake are evident in Figure 35 as near vertical deflections or "kinks" in the diagonal lines. Shortening of the EDM lines due to the Morgan Hill earthquake ranged from about 0.30 in. to 0.63 in. The impact on earthquake-induced slide movement due to the Loma Prieta earthquake may not have been as significant as the Morgan Hill earthquake. As shown on Table 3, good pre-and post-survey data exists only for survey point D-1 which showed a deflection of about 0.36 in. This is about the same deflection for the same survey point as occurred during the Morgan Hill earthquake. For survey points A and C, however, the necessary survey readings to assess the magnitude of earthquake-induced settlement for the Loma Prieta are not available. Nevertheless, using pre- and post-average rates, we estimate that survey points A and C moved about 0.1 and 0.4 inch, respectively, due to the Loma Prieta earthquake. As pointed out in Section 3.5, the two earthquakes are estimated to have produced significantly different peak ground acceleration values at the site; 0.18 g for the Morgan Hill earthquake and 0.10g for the Loma Prieta earthquake. In assessing earthquake-induced movement one must also consider the duration of strong shaking which is defined as the elapsed time in seconds between the first and last occurrence of a specified acceleration value, such as 0.05g. For example, the duration of strong shaking for the Morgan Hill event was 5-6 seconds, while the duration for the Loma Prieta event was about 15 seconds. In comparing the earthquake-induced landslide movement for these two earthquakes, it would appear that the Morgan Hill produced slightly more landslide movement than the Loma Prieta earthquake. It also appears that earthquake-induced movements within the landslide mass can vary significantly even for the same earthquake.

6.5 Time Domain Reflectometry (TDR)

6.5.1 Methodology of Analyzing TDR Data

Raw TDR signal data are plotted using public-domain software DPLOT95 developed by the U.S. Army Corps of Engineer. The data plots consist of relative reflectance in milli-rhos versus depth in feet. An initial reading of the relative reflectance of the cable is taken soon after the cable is installed. This initial reading is used as a baseline for comparison of subsequent field readings.

As the TDR cable is deformed due to progressive landslide creep along a discrete shear plane the roughly vertical readout signal records a sub-horizontal spike. The depth of the deformation (or spike) and relative length of the spike are determined using the software, which compares the latest data to the initial reading. The DPLOT95 software performs the necessary data comparisons. Data plots based on successive field measurements will show progressively longer spikes at the same depth in the TDR plots. The size and relative length of the spike increases with increased deformation during landsliding. It is possible to detect multiple zones of deformation of the TDR cable, which are evident as multiple spikes at the respective depths at which deformation is taking place.

6.5.2 TDR Results

Figure 37 shows TDR data plots for DWR-6, DWR-7, DWR-8, and I-20. Initial baseline readings were collected from DWR-8 on February 18, 2000, from DWR-6 on April 12, 2000, and from DWR-7 and I-20 on May 23, 2000. The latest reading was collected from all TDR cables on October 18, 2000. Except for a spike at a depth of about 118 feet in DWR-7, no obvious deformation of the TDR cables is evident on the data plots. The spike at 118 feet in DWR-7 appears to have increased in length during the period from May 23, 2000 (initial reading) to October 18, 2000. This depth does not correlate to the expected depth of the main slide plane beneath DWR-7. The depth could however, correspond to a secondary creep zone. We will continue to monitor the spike for indications of movement.

6.6 Groundwater Conditions

The District has installed 26 piezometers in 14 borings within the landslide. Readings of the piezometers are generally taken quarterly. A complete discussion of the piezometer locations, tip depths, piezometer types, detailed readings, and other pertinent aspects of the piezometers is presented in Appendix B. There are some interesting features or characteristics in some of the plots presented in Appendix B that should be noted. In Figures B-10 through B-16, the readings start out high (or low for P-16 in Figure B-11) and then reach equilibrium after one to three years. The high initial readings probably represents water being introduced into the formation from the drilling operations. The decline in piezometric level over time represents the time it takes for the clays to give up their water introduced during drilling and reach equilibrium with the surrounding formation. The potential implication of these observed time delays is further discussed later in this section.

Based on a reasonable interpretation of the data from the 26 piezometers, it appears that two independent piezometric surfaces are being monitored at the site. The first and highest of the two surfaces represents pore pressures acting within the landslide. The second and lower of the two surfaces represents pore pressures acting below the landslide surface. These two independent piezometric surfaces are further discussed below.

6.6.1 Piezometric Conditions Within the Landslide

Contours of equal piezometric head are presented in Figure 38 for the portion of the landslide within or adjacent to District property. The contours were drawn using a plotting program (Surfer) that integrates between discrete points and then produces smooth contours to fill the two dimensional area containing the discrete data points. The data used to compute the piezometric contours are from September 1999. As shown in Figure 38, the piezometric surface drops from about el. 450 above the terminal reservoir to about el. 260 just below the Penvault. The average seepage gradient (head loss per unit length in the direction of flow) is between 0.067 and 0.14 ft/ft. It will also be noted that the seepage gradients are generally normal to the surface slope (i.e. piezometric contours essentially parallel to the ground surface). Although the piezometric contours are relatively smooth, there does appear to be a squeezing of contours just downstream of the Terminal Reservoir. This suggests that the Terminal Reservoir may have been leaking for some unknown period of time prior to its recent (spring 1999) refurbishment. This interpretation is inferred solely from the response of piezometers P-5 and P-6 (see Appendix B), which are the two closest piezometers located in a single drill hole less than 100 feet from the west edge of the tank. Readings from these two piezometers were stopped in mid-1987, and then resumed in early 1995. P-6 has its tip at 413.6 feet, or 38.5 feet below ground surface. P-5 has its tip at el. 382.1, or 70.0 feet below ground surface. In comparing the pre-1987 and post-1995 readings from these two piezometers, it appears that the average value of piezometric heads for P-5 and P-6 increased about 9 feet and 17 feet, respectively. It is not clear whether leakage from the Terminal Reservoir was responsible for the rise in piezometric head. Now that the Tank has been repaired, the future values of these piezometers should be monitored to determine if the trend reverses. It appears that the last data point in the fall of 2000 (PSE = 435) suggests that piezometric head for both of these piezometers has dropped slightly when compared to similar data from 1998 and 1999. Clearly, a more reliable interpretation could be achieved with the installation of more piezometers in the area of the Terminal Reservoir.

During the future completion of the Phase 2 TWIP work, the geotechnical consultant has proposed a concept of increasing the overall factor of safety during construction by constructing a deep well seepage collection system. The final geotechnical investigation report (Harza, 2000) states the following;

> *"We anticipate that relatively high ground water table will be encountered during excavation. We recommend that a contingency be established for dewatering the bottom of the excavations during construction....Due to the high fluctuation of ground water level measured from the nearby piezometers, we recommend that the least two additional piezometers be installed at the Ozone site to measure the ground water level at the specific site location.*

> *During installation of piezometers, continuous soil coring could be performed to investigate the possible presence of shallow slide planes...In addition, field and/or laboratory permeability tests may be conducted to determine the permeability of on-site soils. These efforts would help design the most cost-effective system for temporary and permanent dewatering at the Ozone site, in addition to minimizing the uncertainty in dealing with ground water level during the bidding process".*

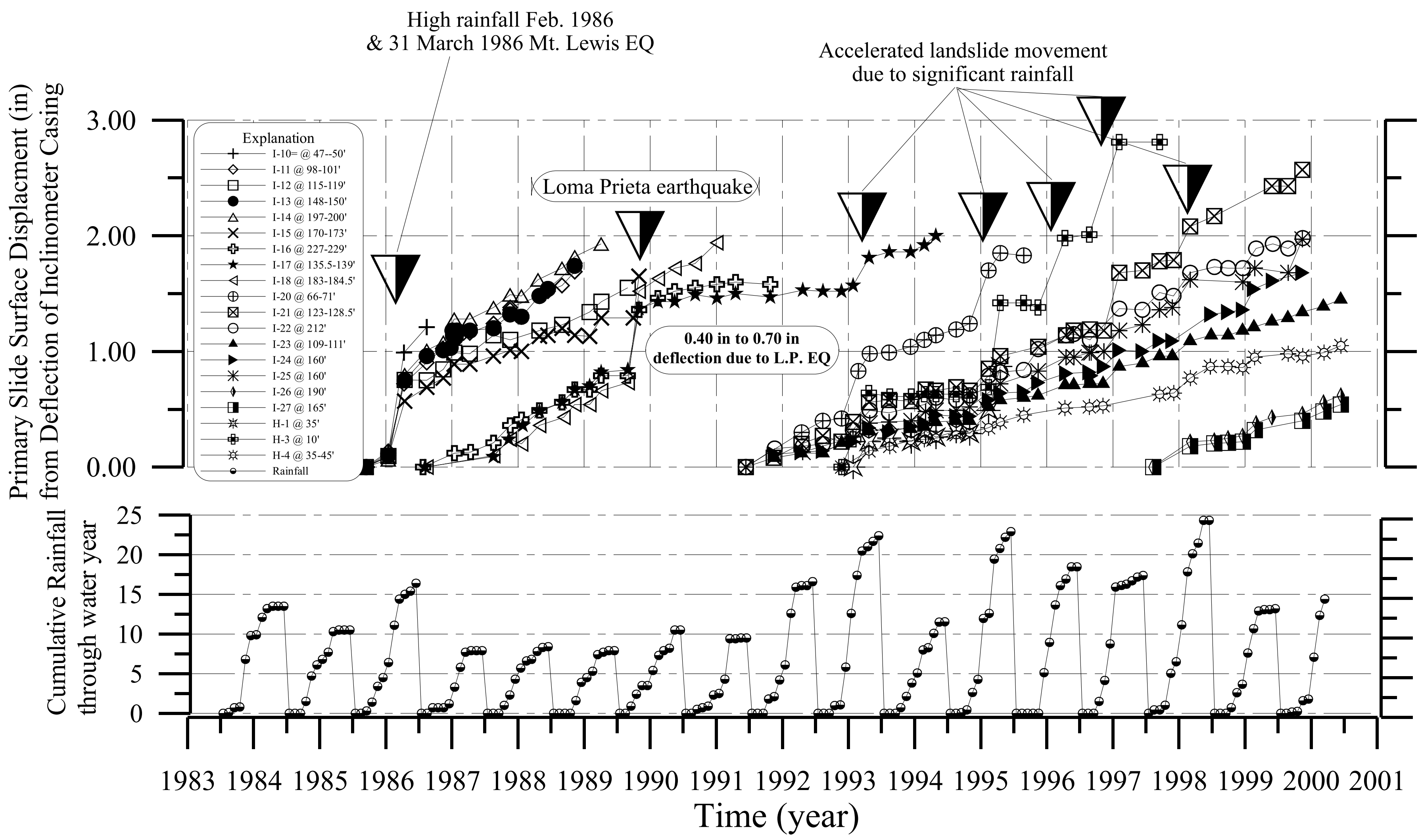
Our review of field conditions that will impact the Stage 2 TWIP construction suggests that groundwater most likely will be encountered during the excavation for the Ozone Generation building and Ozone contactor basins.. These structures will have the bottom of the slab at about el. 415 feet, which is between 18 feet and 31 feet below ground surface along the base of the slab. Additionally, construction of the contactor basins will include pipe and sump invert elevations 10 to 15 feet below the slab. In a worse case scenario, if the piezometric conditions existing at PZ-3 (P-5 and P-6) is projected about 150 feet downhill, the piezometric surface elevation could be at el. 431 feet, thus suggesting up to 16 feet of water pressure at the base of the slab. We concur that additional piezometers (most likely open well types) should be placed around the planned excavation several months before the planned excavation in order to determine a more precise distribution of piezometric conditions. Depending on the results, it may be appropriate to consider a permanent dewatering system as recommended by the Consultant.

Initially, it was thought that an intermediate piezometric surface could also interpreted from the piezometer data as shown in Figure 39. The intermediate piezometric surface was thought to appear to represent pore pressures that are between 45' and 125' lower than the shallowest piezometric surface discussed above. As shown in Figure 39, the gradient of head loss is approximately parallel to the ground surface and about 90 degrees from that of the shallowest piezometric surface shown in Figure 38, suggesting that the intermediate piezometric surface is somehow influenced by the presence of the ancestral Dutard Creek. Although the pore pressures used to contemplate an intermediate piezometric surface represent actual data, it was concluded that that the hypothesized intermediate piezometric surface most likely was being influenced by leakage of pore pressures caused only at specific locations by nearby inclinometer borings that penetrate the base of the landslide. The pore pressures acting below the slide plane in the more permeable underlying alluvial materials are discussed below.

6.6.2 Piezometric Conditions Acting Below the Landslide

One of the unexpected responses detected by some of the piezometer and inclinometer readings is the presence of a highly permeable material below the failure plane. As discussed in Chapter 3, the material underlying the lower portion of the slide is considerably coarser grained than the landslide mass and is thought to be the ancestral valley fill material (older alluvium). During the installation of inclinometer DWR-6, which is located adjacent of the Terminal Well, DWR encountered extremely granular material for about 100 feet beneath the landslide. At least a portion of this material may be representative of coarse grained stream deposits associated with the ancestral Dutard Creek. Several piezometers and inclinometers have penetrated the base or failure plane of the landslide. These instruments include piezometer PZ-14 and inclinometers I-21, I-22, I-23, and I-26. Using the piezometric data from these instruments, a plot of contours of equal piezometric head representative of the materials under the slide was computed. The results presented in Figure 40 suggest a significantly lower piezometric surface acting within the alluvial materials underlying the slide compared with the slidemass material. This groundwater anomaly is further discussed in Section 7.6.5.

Finally, as noted at the beginning of this section, based on the performance of several piezometers after installation, a question can be raised about the reliability of most piezometer data from pneumatic piezometers and how quickly the piezometers take to respond or reflect a change in piezometric conditions. If it takes one to three years for the effects of drilling to dissipate, it seems implausible that seasonal fluctuations over one year's time can be detected by the pneumatic piezometers. Theoretically, if the formation is completely saturated, the pore pressure changes should be transmitted virtually instantaneously. It appears likely that response times may be significantly improved with the use of vibrating gauge piezometers.



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig21a.grf

DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

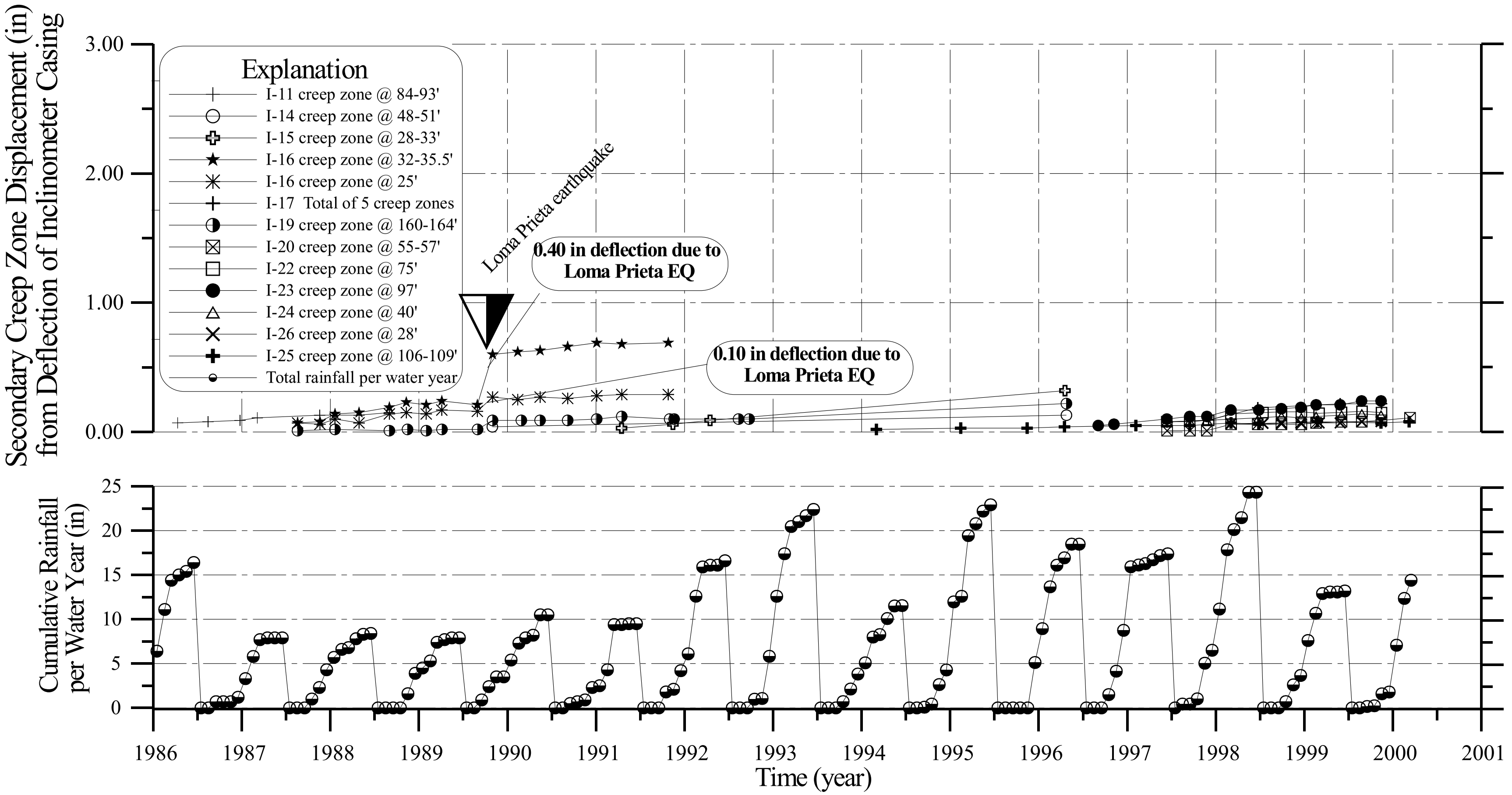
Santa Clara Valley Water District

Primary Slide Surface Displacement Versus Time
from Deflection of Inclinometer Casing

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
none

FIGURE
21



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig22.grf

DATE 10/5/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

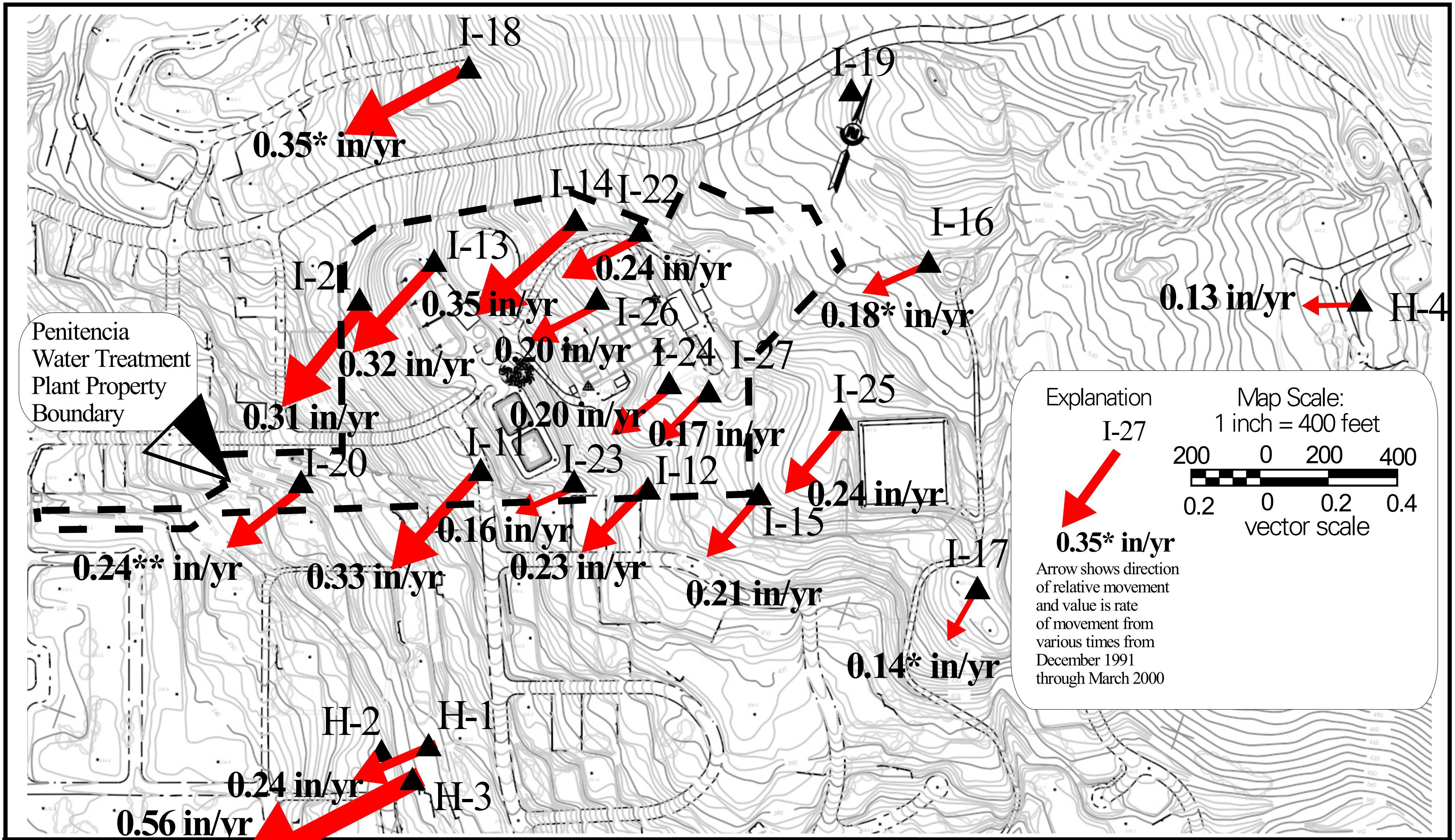
Secondary Creep Zone Displacement Versus Time from Deflection of Inclinometer Casing

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE

FIGURE 22

Penitencia Creek Landslide



Notes: 1) Base map by Transvision,
dated 2/25/00, elevations based on NAVD 1988 datum,
photography by Air Flight Service, flown 12/99
2) File W:\geology\penitencia creek landslide\figures\fig23a.grf
3) * = corrected for accelerated movement due to wet winter

DATE	10/5/00
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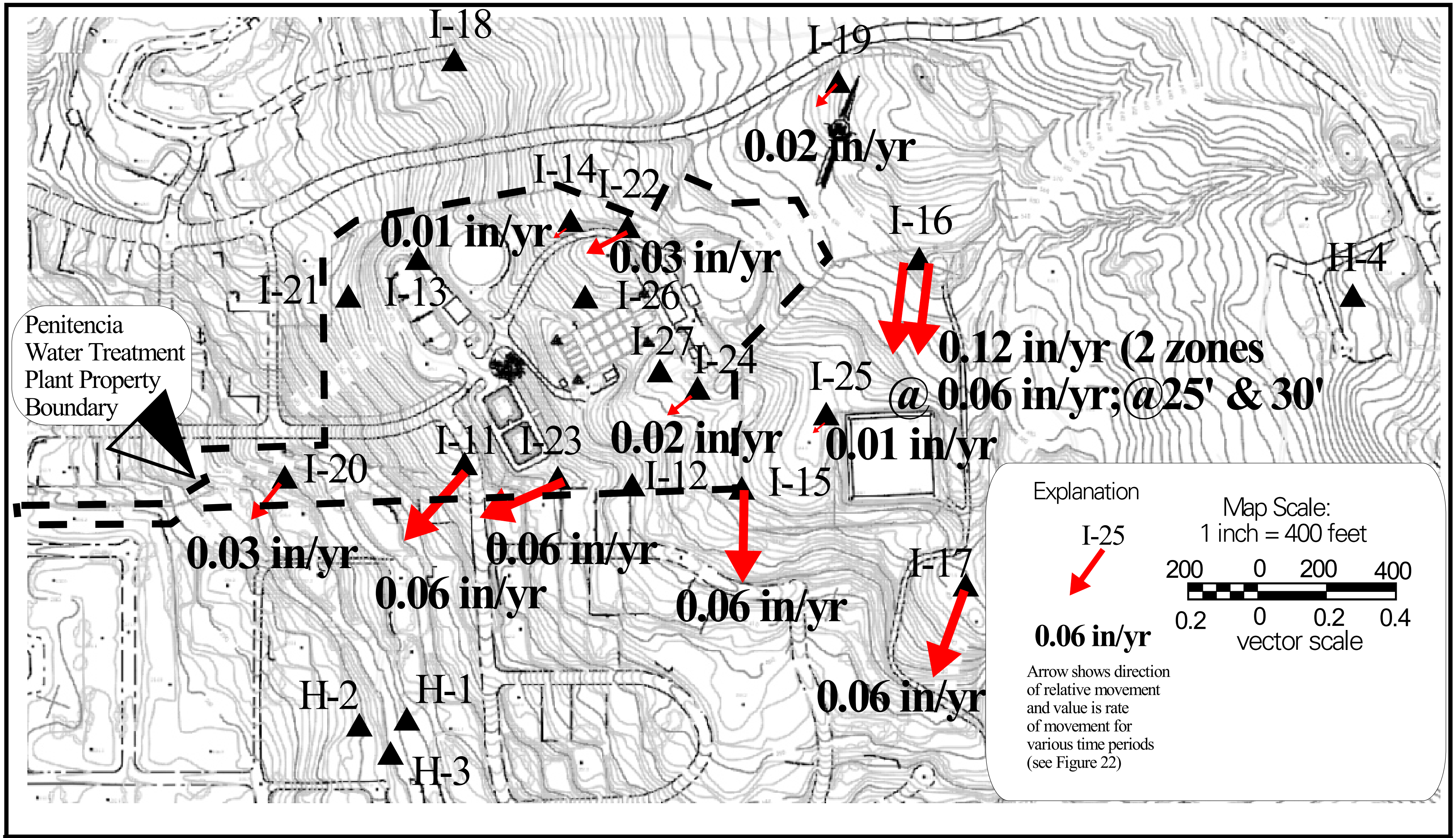
Santa Clara Valley Water District

Displacement Vectors of Primary Slide Surface
Based on Inclinometer Deflection Data
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=400'

FIGURE
23

Penitencia Creek Landslide



Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum, photography by Air Flight Service, flown 12/99
2) File W:\geology\penitencia creek landslide\figures\fig24a.grf
3) * = corrected for accelerated movement due to wet winter

DATE 10/5/00
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DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

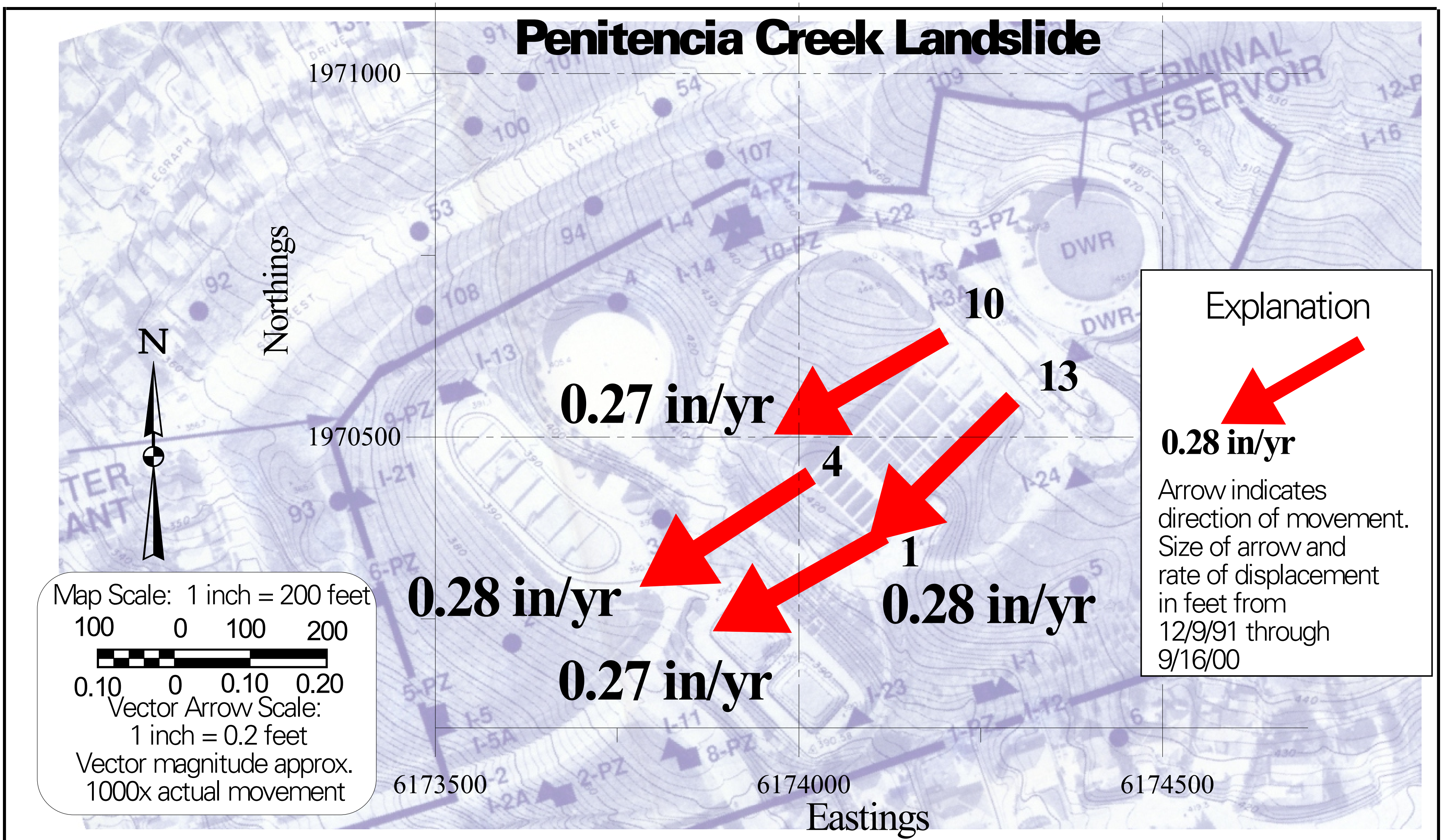
Displacement Vectors of Secondary Creep Zones
Based on Inclinometer Deflection Data

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=400'

FIGURE 24

Penitencia Creek Landslide



Explanation

0.28 in/yr

Arrow indicates direction of movement. Size of arrow and rate of displacement in feet from 12/9/91 through 9/16/00

Map Scale: 1 inch = 200 feet
100 0 100 200

0.10 0 0.10 0.20
Vector Arrow Scale:
1 inch = 0.2 feet
Vector magnitude approx. 1000x actual movement

Note: 1) W:\geology\penitencia creek landslide\figures\fig25.grf
2) gridlines are California Coordinates, horizontal datum NAD 83

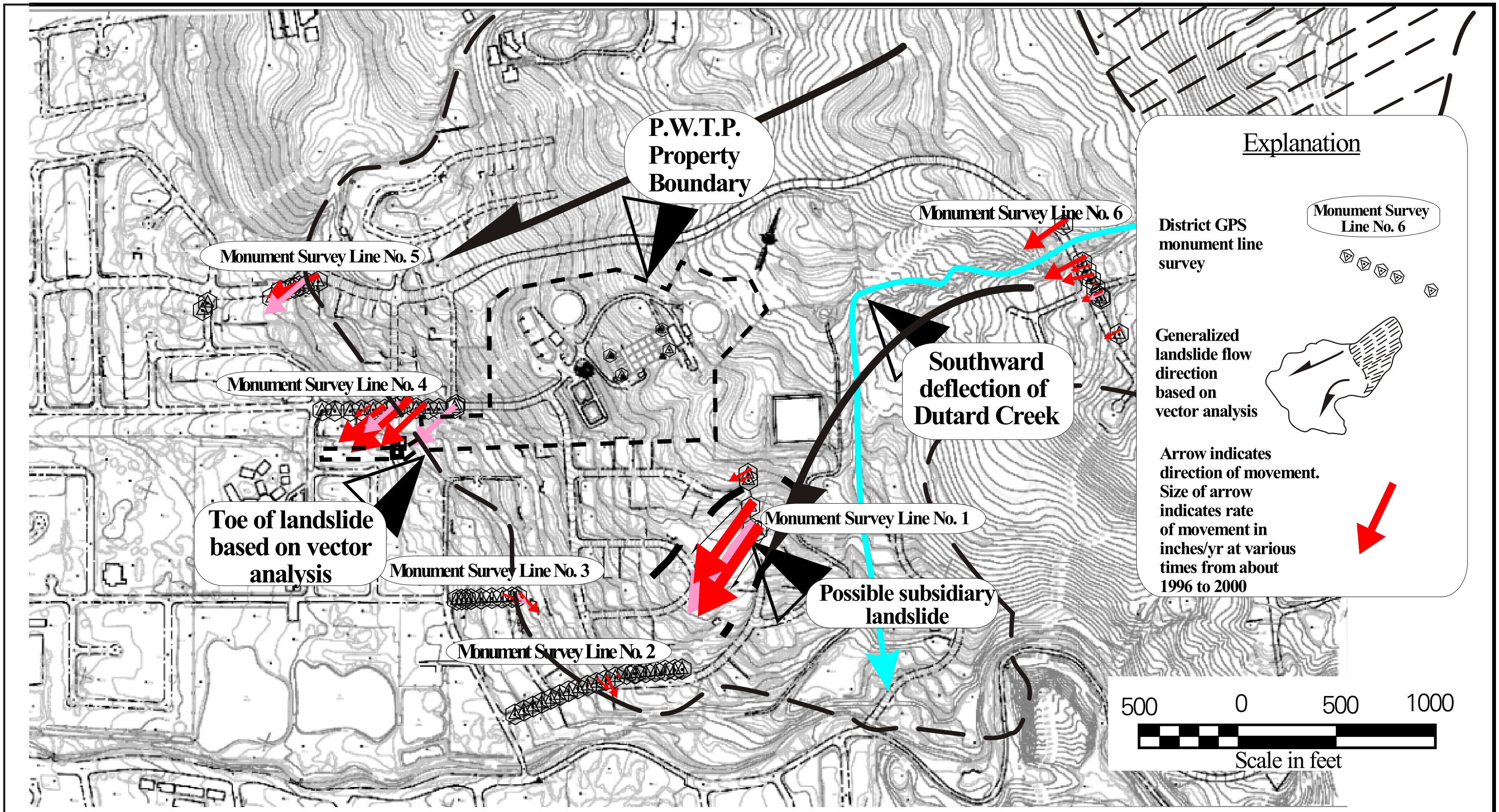
DATE	12/11/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Displacement Vectors
Based on Plant Monument
Survey Stations 1, 4, 10, 13
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=200

FIGURE
25


P.W.T.P. Property Boundary
Monument Survey Line No. 5
Monument Survey Line No. 4
Toe of landslide based on vector analysis
Monument Survey Line No. 3
Monument Survey Line No. 2
Monument Survey Line No. 1
Possible subsidiary landslide
Southward deflection of Dutard Creek
Monument Survey Line No. 6
Explanation
District GPS monument line survey
Monument Survey Line No. 6
Generalized landslide flow direction based on vector analysis
Arrow indicates direction of movement. Size of arrow indicates rate of movement in inches/yr at various times from about 1996 to 2000
500
0
500
1000
Scale in feet

Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum, photography by Air Flight Service, flown 12/99

2) File W:\geology\penitencia creek landslide\figures\fig26.grf

3) * = corrected for accelerated movement due to wet winter

DATE 09/27/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

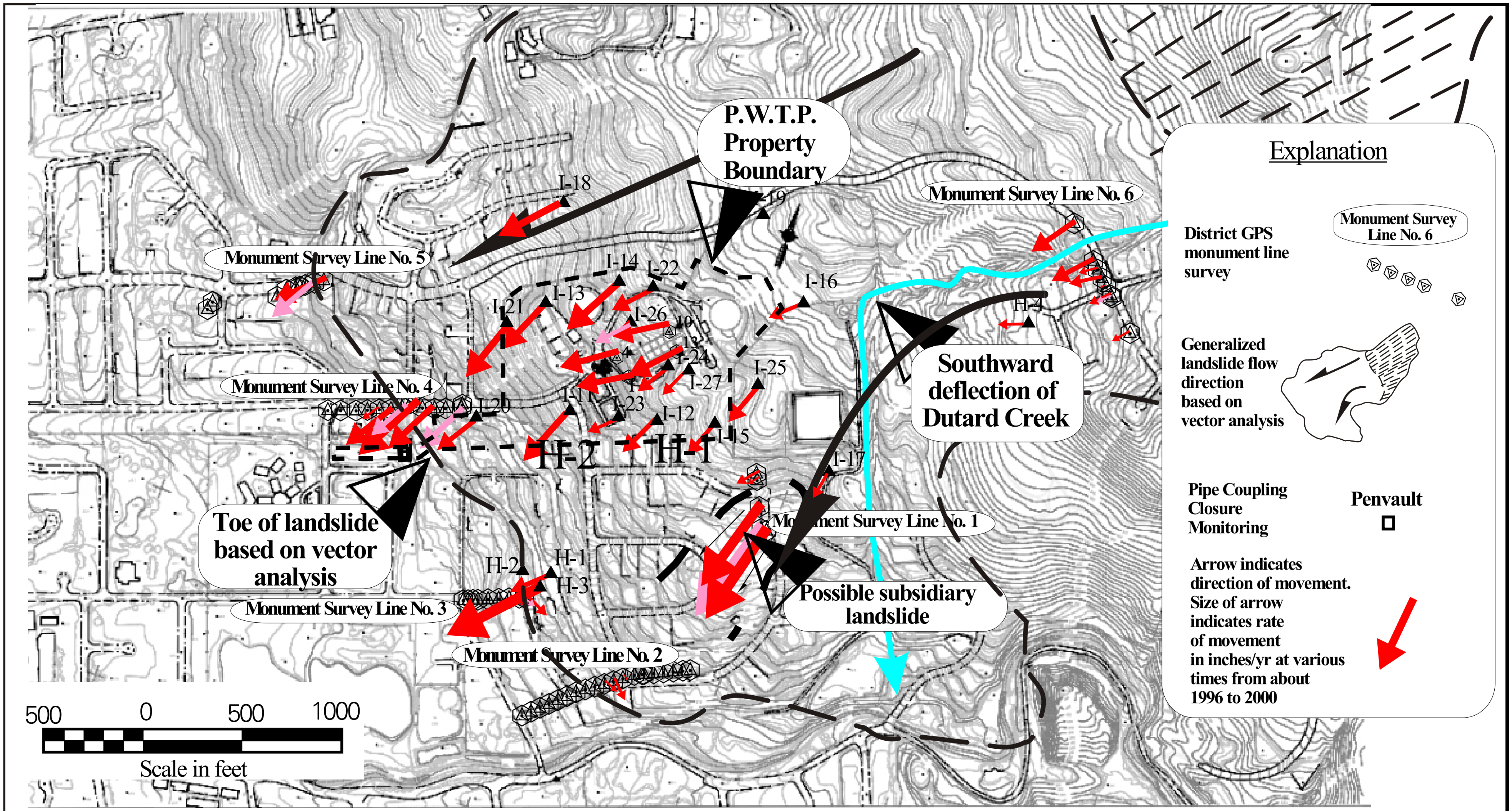
Santa Clara Valley Water District

Vector Map of Monument Line Surveys

Penitencia Water Treatment Plant
Penitencia Creek Landslide

SCALE 1"=500'

FIGURE 26



Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum, photography by Air Flight Service, flown 12/99
2) File W:\geology\penitencia creek landslide\figures\fig27a.grf
3) * = corrected for accelerated movement due to wet winter

DATE 09/27/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

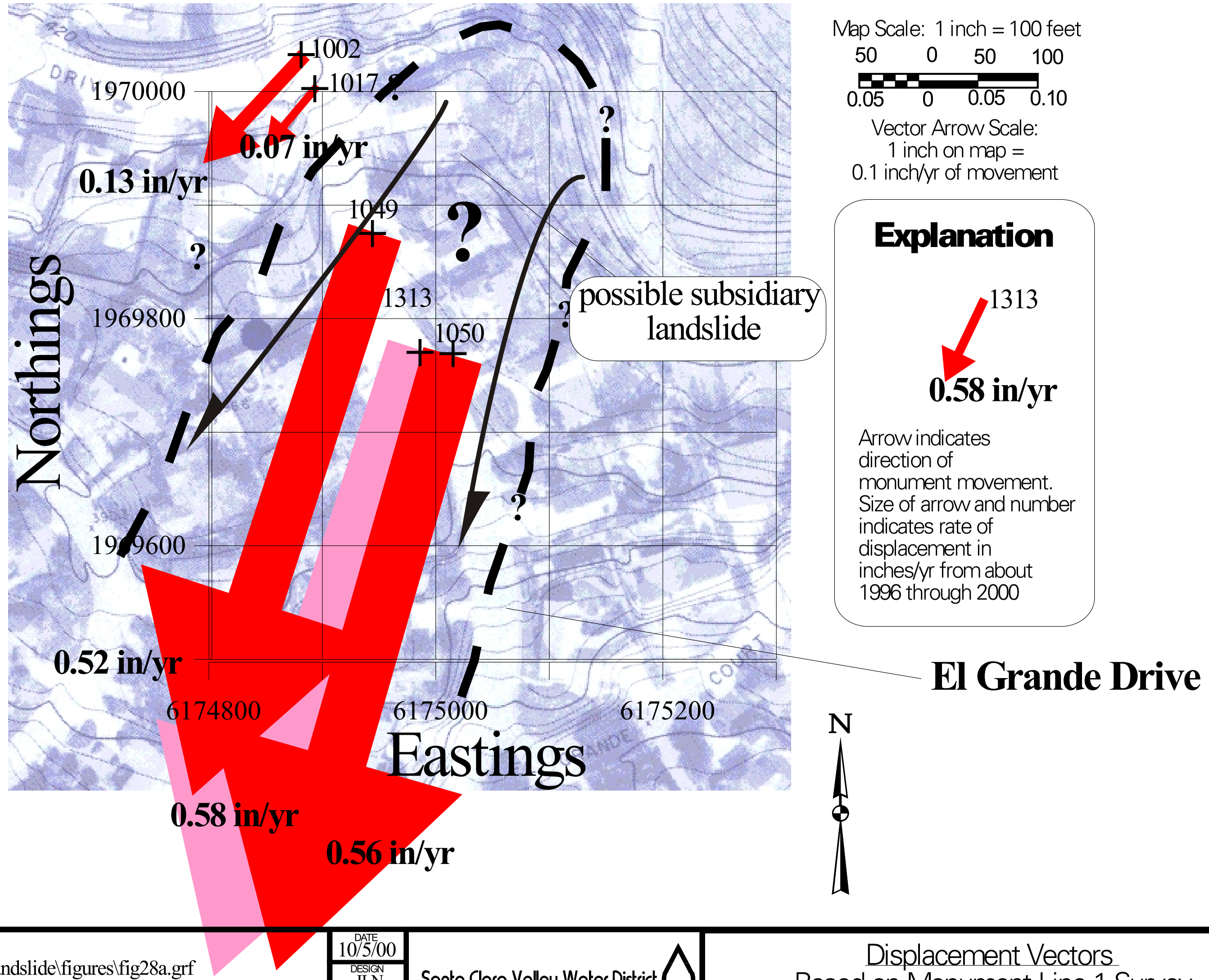
Composite Vector Map of Inclinometers and Monument Line Surveys

Penitencia Water Treatment Plant
Penitencia Creek Landslide

SCALE 1"=500'

FIGURE 27

Penitencia Creek Landslide
Plan View of Monitoring Line 1


1002
1017
0.13 in/yr
0.07 in/yr
1049
1313
1050
?
possible subsidiary landslide
0.52 in/yr
0.58 in/yr
0.56 in/yr
1970000
1969800
1969600
Northings
6174800
6175000
6175200
Eastings
El Grande Drive
N
Map Scale: 1 inch = 100 feet
50 0 50 100
0.05 0 0.05 0.10
Vector Arrow Scale: 1 inch on map = 0.1 inch/yr of movement
Explanation
1313
0.58 in/yr
Arrow indicates direction of monument movement. Size of arrow and number indicates rate of displacement in inches/yr from about 1996 through 2000

Notes: 1) File W:\geology\penitencia creek landslide\figures\fig28a.grf
2) gridlines are California Coordinates, horizontal datum NAD 83
3) pink color used as contrast

DATE	10/5/00
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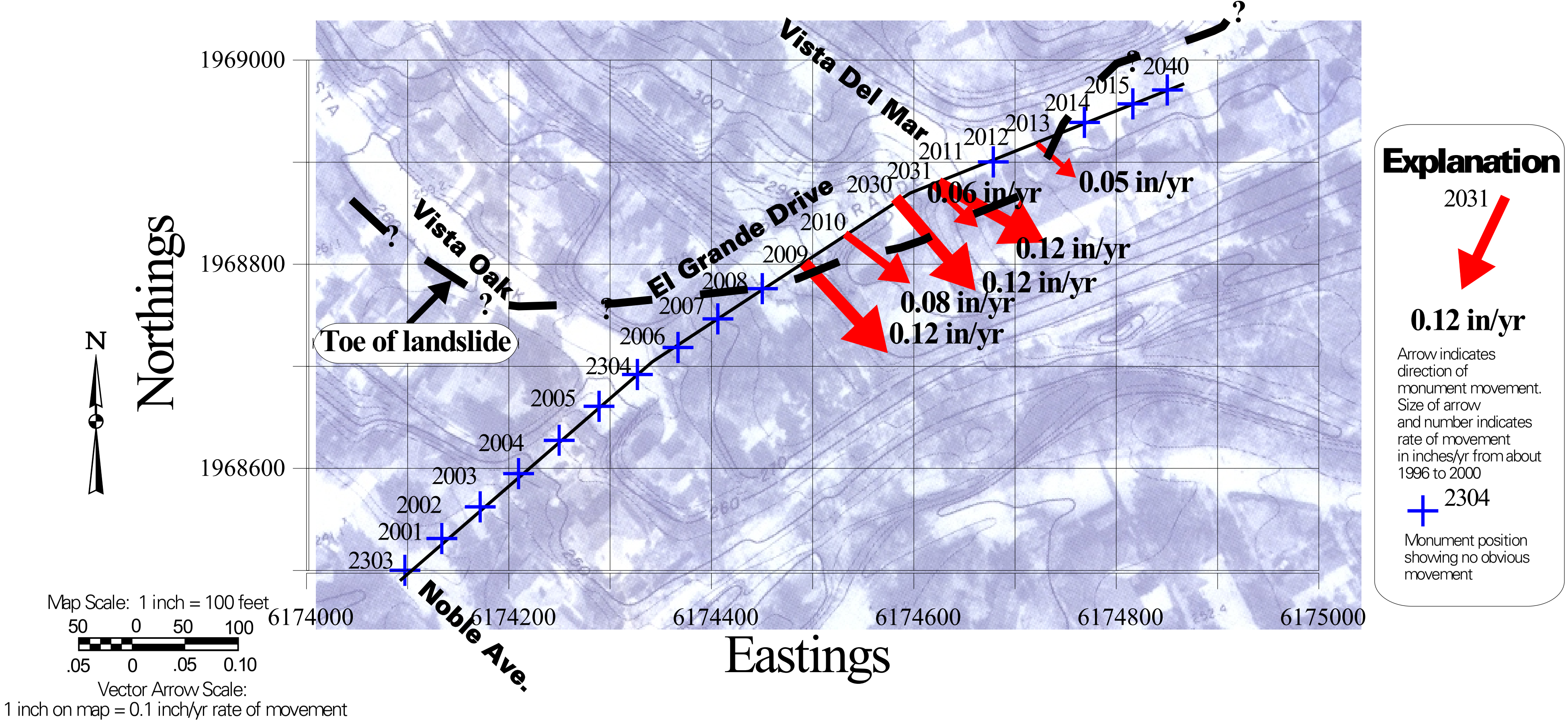
Santa Clara Valley Water District

Displacement Vectors
Based on Monument Line 1 Survey
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=100'

FIGURE
28

Penitencia Creek Landslide
Plan View of Monitoring Line 2



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig29a.grf
2) gridlines are California Coordinates, horizontal datum NAD 83

DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Displacement Vectors
Based on Monument Line 2 Survey
Penitencia Creek Landslide
Penitencia Water Treatment Plant

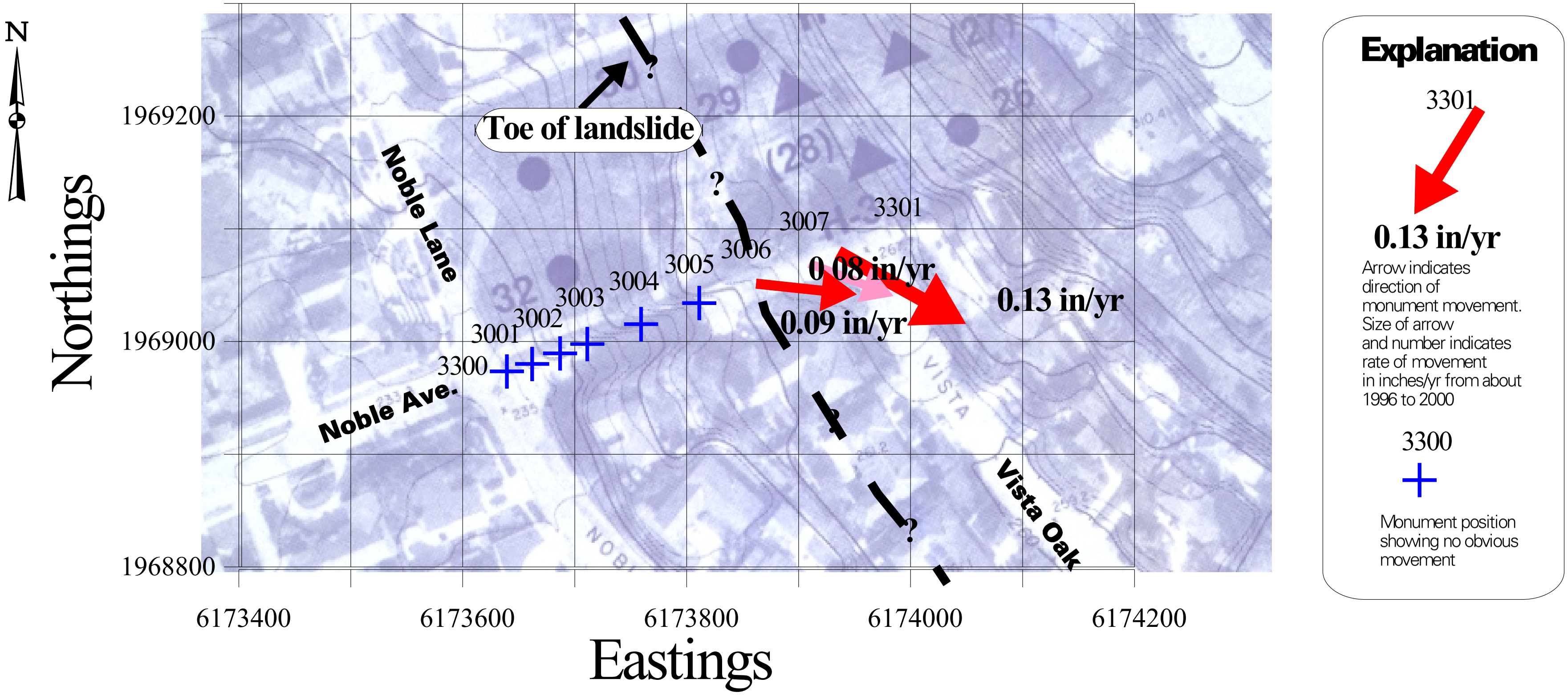
SCALE
1"=100'

FIGURE
29

Penitencia Creek Landslide

Plan View of Monitoring Line No. 3

Map Scale: 1 inch = 100 feet
50 0 50 100
0.05 0 0.05 0.10
Vector Arrow Scale:
1 inch on map = 0.1 in/yr rate of movement



Arrow indicates direction of monument movement. Size of arrow and number indicates rate of movement in inches/yr from about 1996 to 2000

Monument position showing no obvious movement

Notes: 1) File W:\geology\penitencia creek landslide\figures\fig30a.grf
2) gridlines are California Coordinates, horizontal datum NAD 83
3) pink color used for contrast

DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

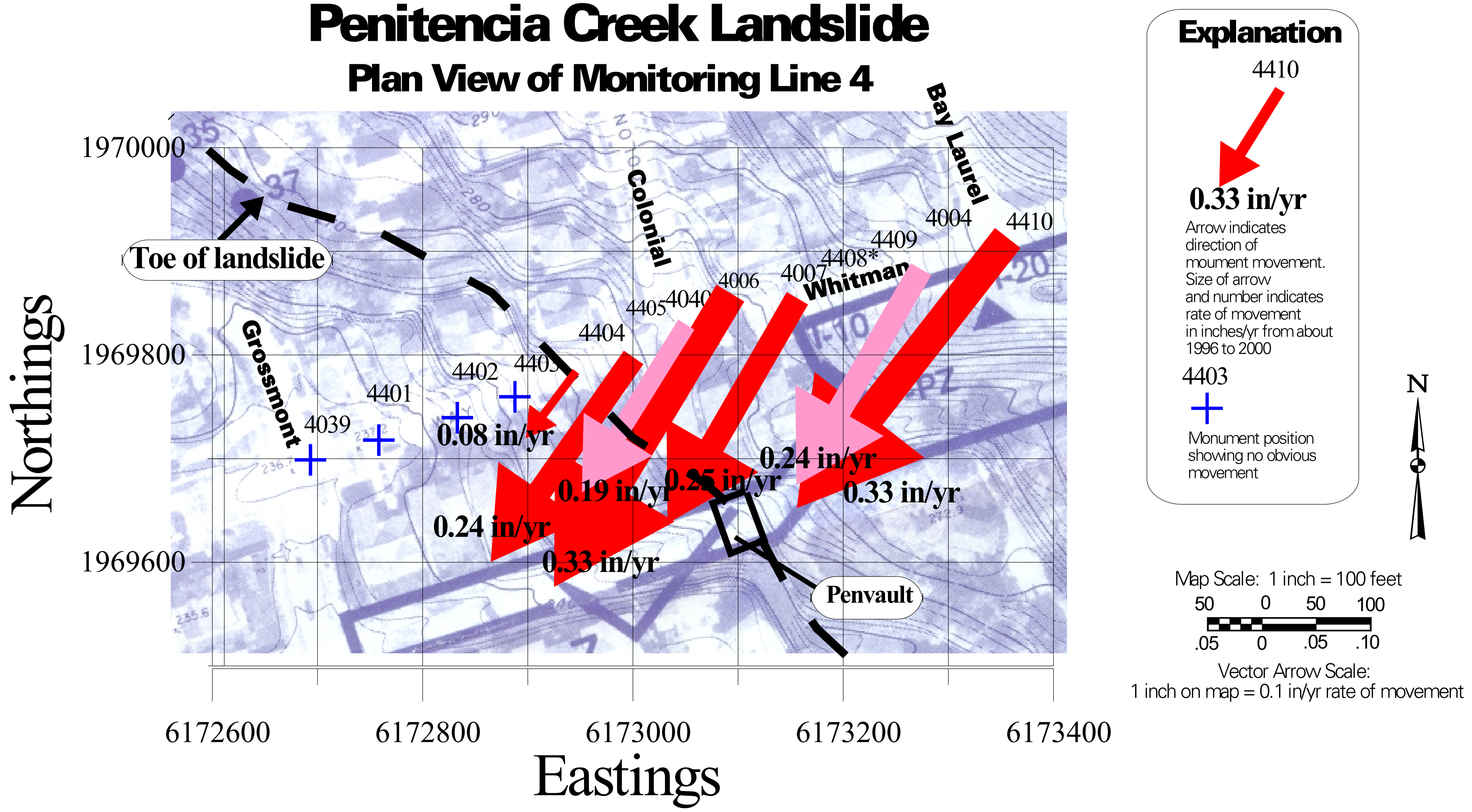
Displacement Vectors
Based on Monument Line 3 Survey
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=100'

FIGURE 30

Penitencia Creek Landslide

Plan View of Monitoring Line 4



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig31a.grf
2) gridlines are California Coordinate, horizontal datum NAD 83
3) * = data from mon. 4408 is spurious and therefore not shown
4) pink color used for contrast

DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Displacement Vectors
Based on Monument Line 4 Survey

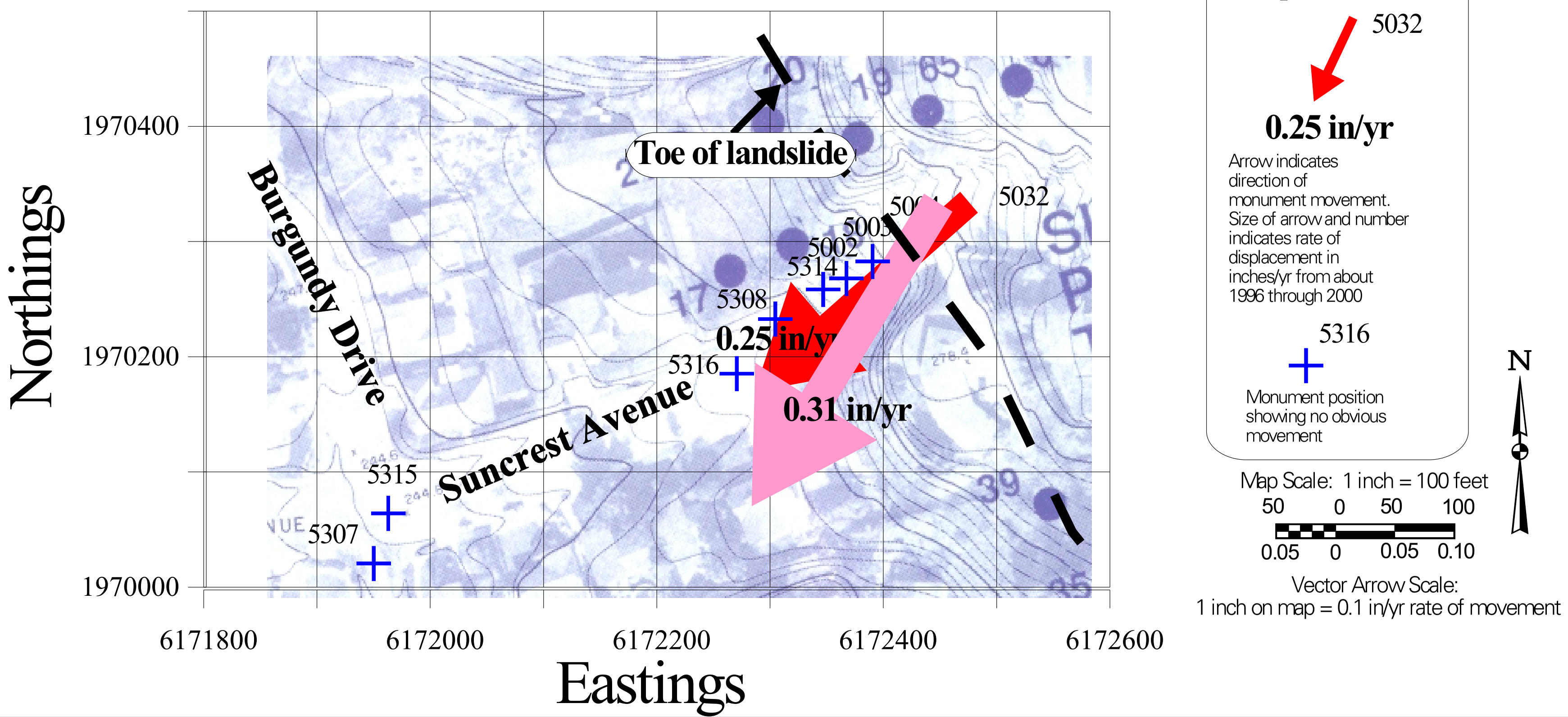
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=100'

FIGURE
31

Penitencia Creek Landslide

Plan View of Monitoring Line No. 5



Northings
1970400
1970200
1970000
6171800
6172000
6172200
6172400
6172600
Eastings
Toe of landslide
Burgundy Drive
Suncrest Avenue
5315
5307
5316
5308
5314
5002
5003
50
5032
0.25 in/yr
0.31 in/yr
Explanation
5032
0.25 in/yr
Arrow indicates direction of monument movement. Size of arrow and number indicates rate of displacement in inches/yr from about 1996 through 2000
5316
Monument position showing no obvious movement
N
Map Scale: 1 inch = 100 feet
50
0
50
100
0.05
0
0.05
0.10
Vector Arrow Scale:
1 inch on map = 0.1 in/yr rate of movement

Notes: 1) File W:\geology\penitencia creek landslide\figures\fig32a.grf
2) gridlines are California Coordinates, horizontal datum NAD 83
3) pink color used for contrast

DATE 10/5/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

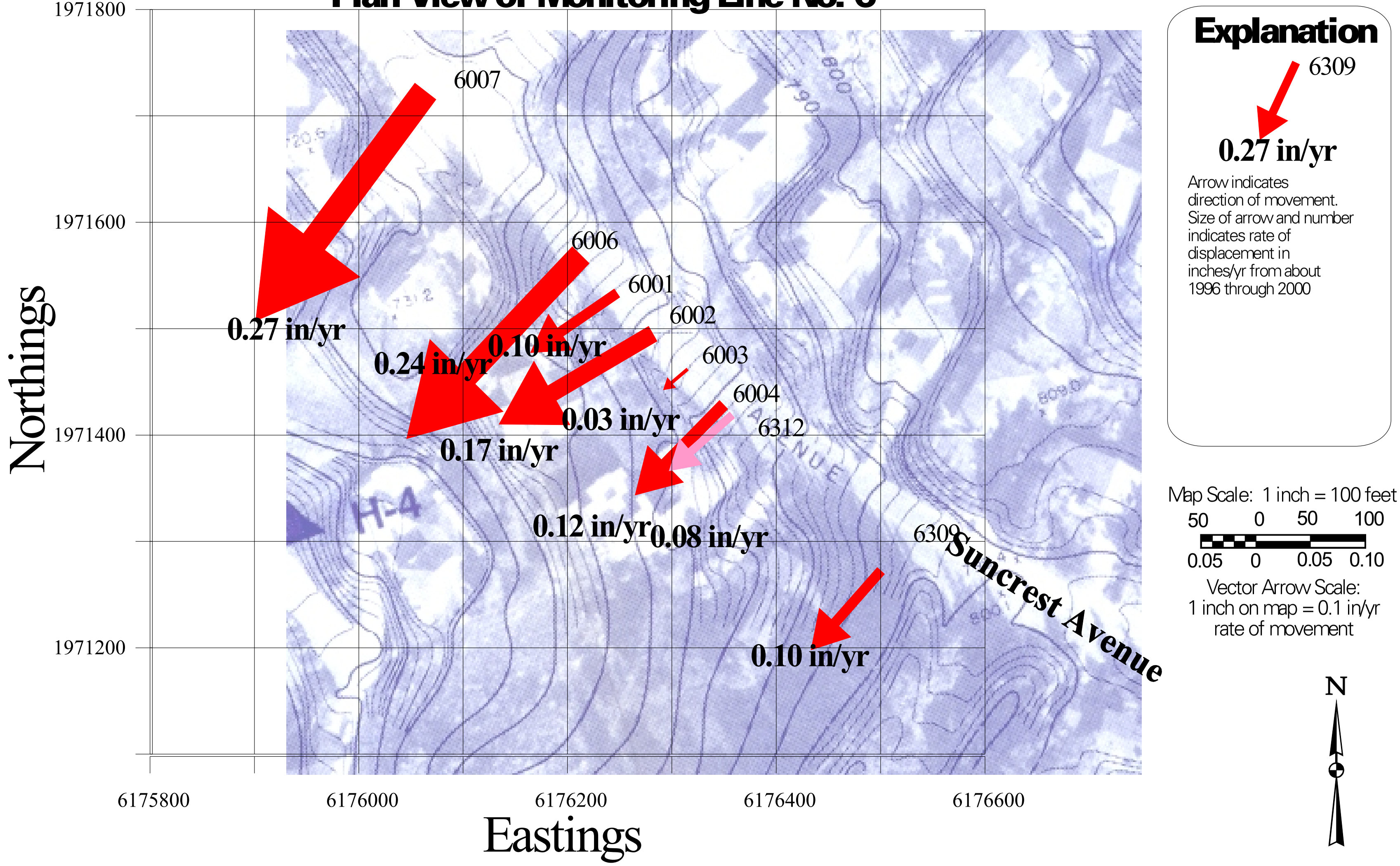
Santa Clara Valley Water District

Displacement Vectors
Based on Monument Line 5 Survey
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=100'
FIGURE 32

Penitencia Creek Landslide

Plan View of Monitoring Line No. 6



Notes: 1) File W:\geology\nelson\penitencia creek landslide\fig33a.grf
2) gridlines are California Coordinates, horizontal datum NAD 83
3) pink color used for contrast

DATE 10/5/00
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Santa Clara Valley Water District

Displacement Vectors
Based on Monument Line 6 Survey

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=100'

FIGURE 33

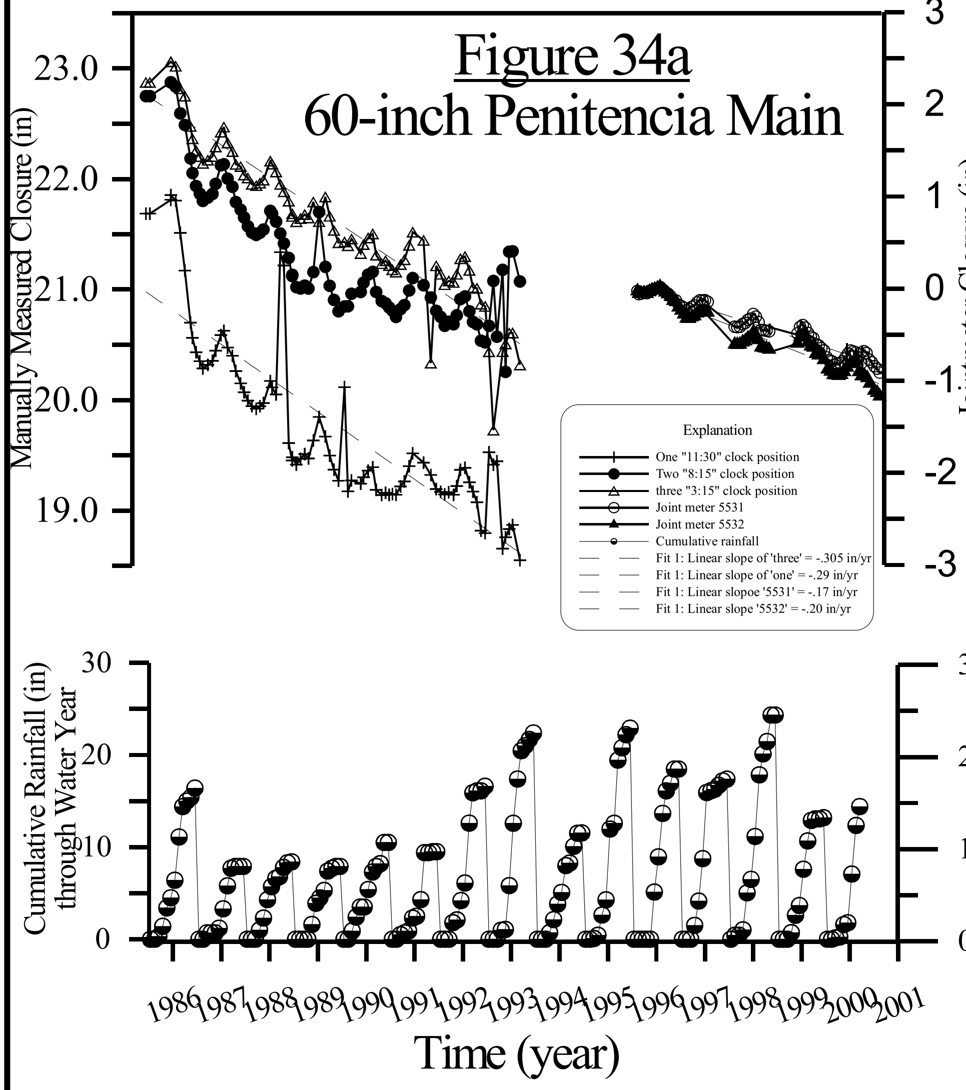


Figure 34a
60-inch Penitencia Main

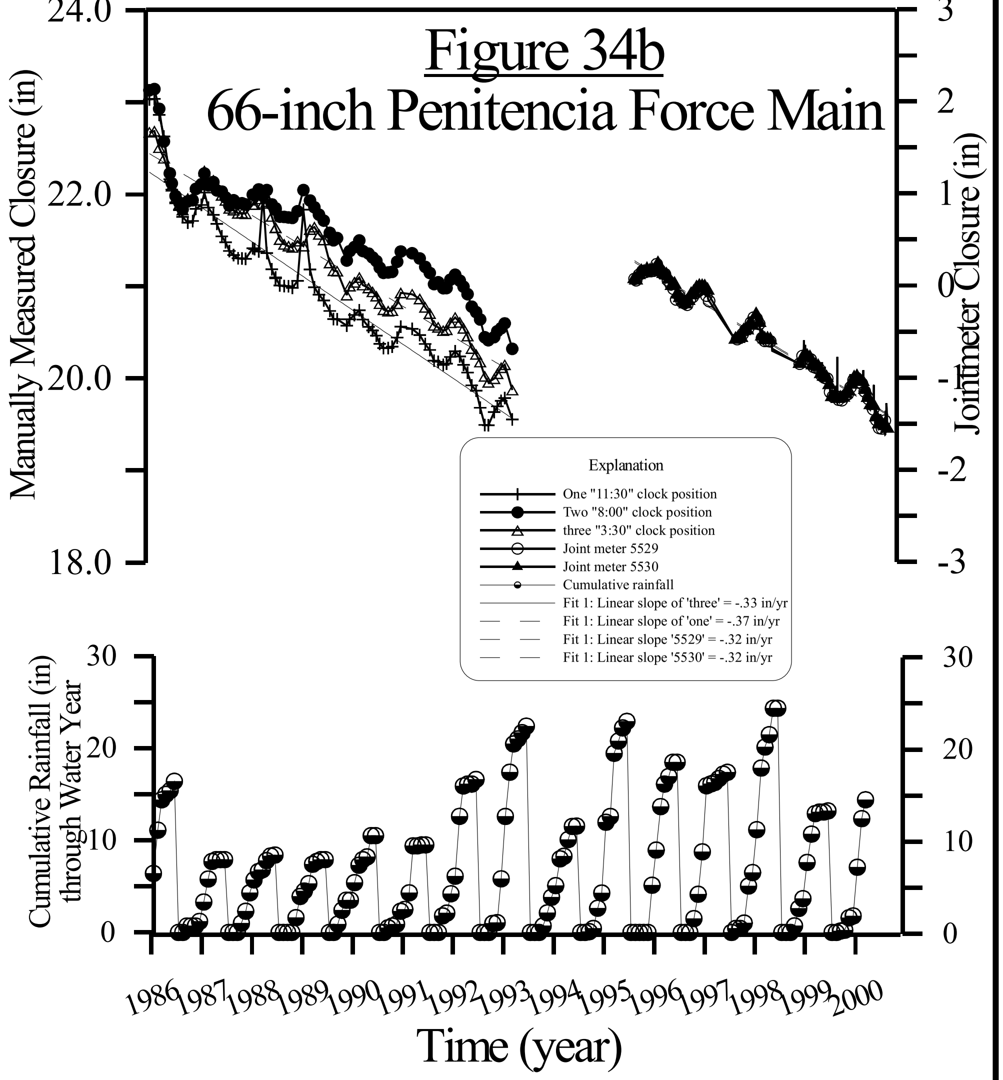


Figure 34b
66-inch Penitencia Force Main

Notes: 1) File W:\geology\penitencia creek landslide\figures\fig34a.grf

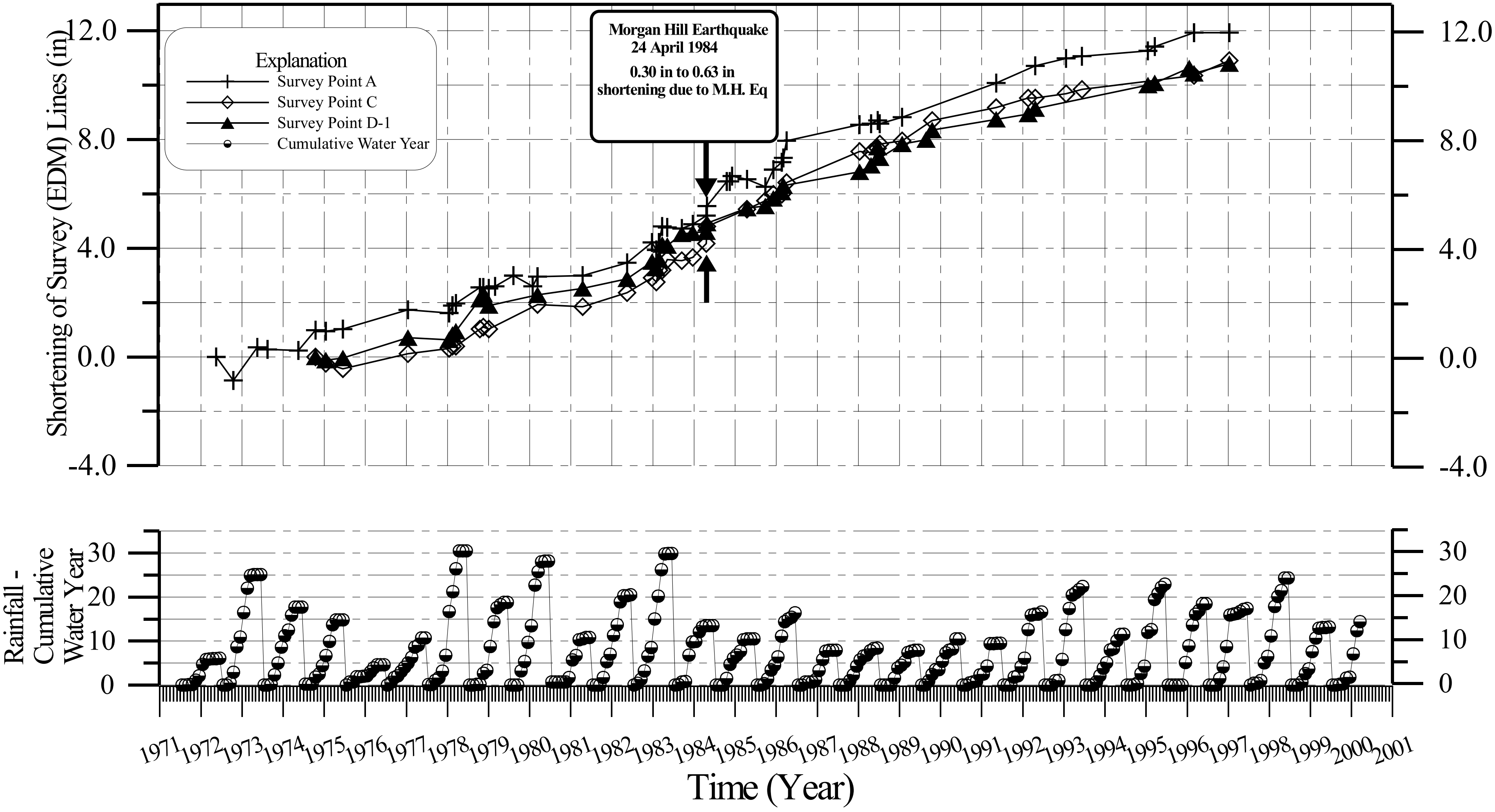
DATE 10/5/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

Pipe Coupling Closure Versus Time
60-inch Penitencia Main
66-inch Penitencia Force Main
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
34



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig35a.grf
2) EDM data supplied by DWR

DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

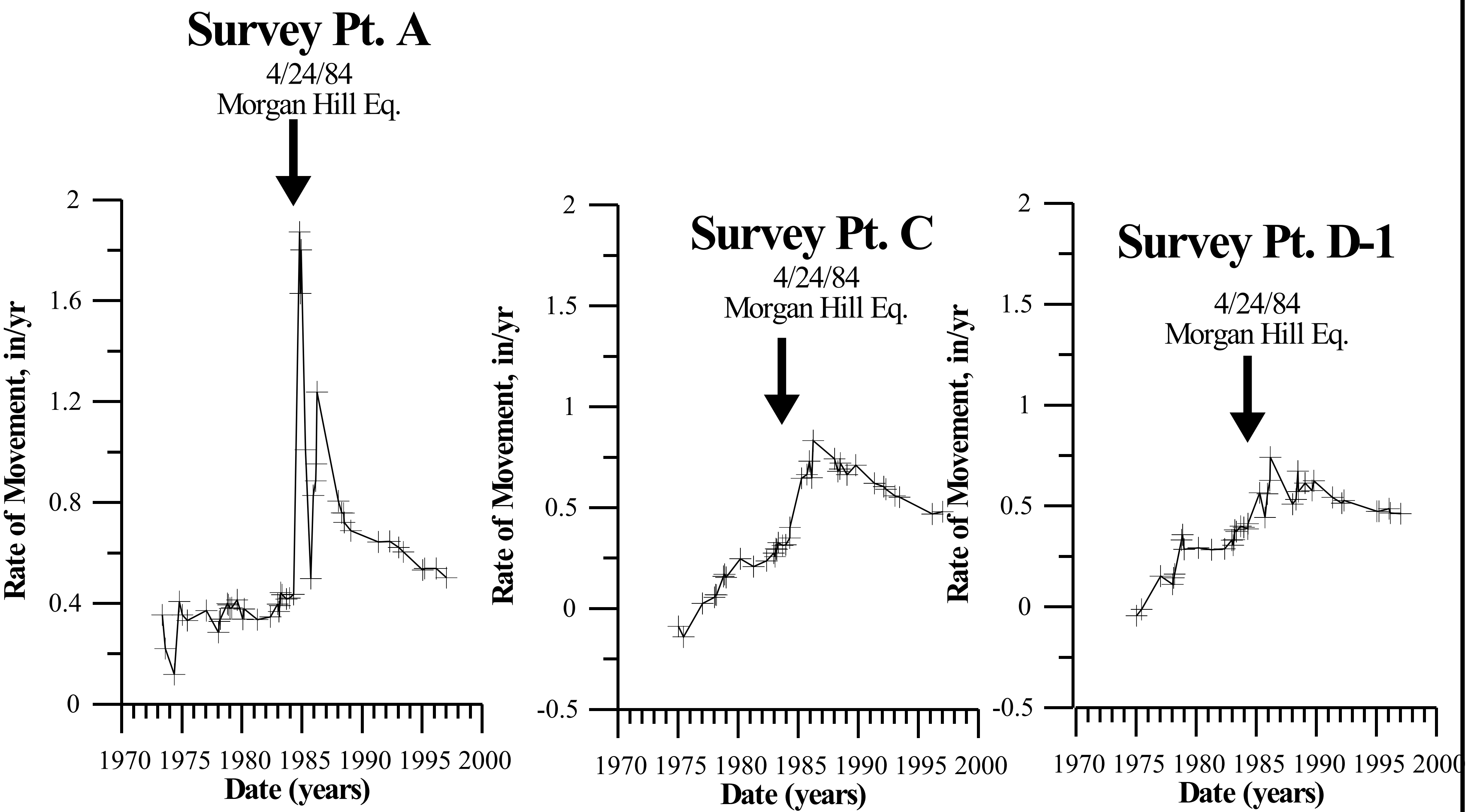
Santa Clara Valley Water District

Shortening of Selected EDM Survey Lines
from ~1972 through 1997

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
35



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig36a.grf

DATE 1/11/01
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

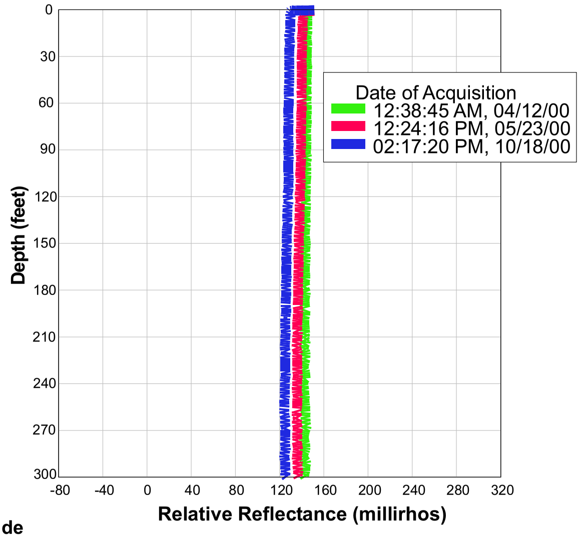
Rate of Movement of Selected EDM
Survey Lines, 1972 through 1997

Penitencia Creek Landslide
Penitencia Water Treatement Plant

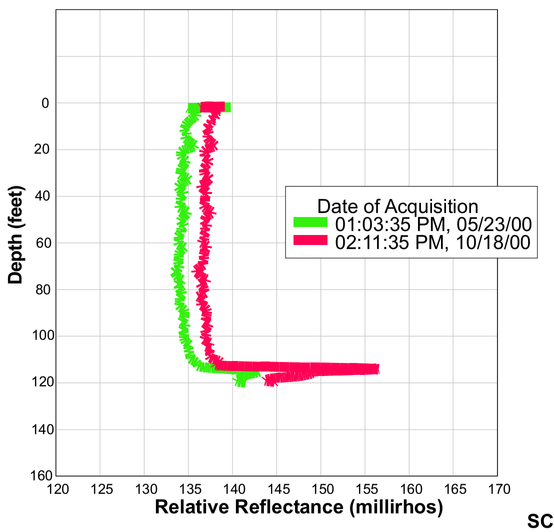
SCALE
as shown

FIGURE
36

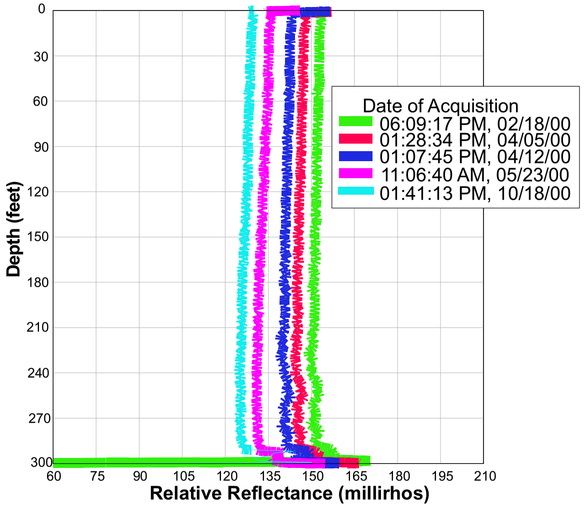
TDR CABLE READING
DWR-6
SCVWD Penitencia Creek Landslide



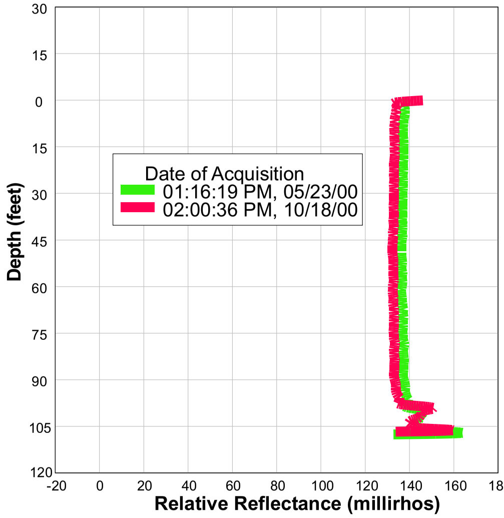
TDR CABLE READING
DWR-7
SCVWD Penitencia Creek Landslide



TDR CABLE READING
DWR-8
SCVWD Penitencia Creek Landslide



TDR CABLE READING
I-20
SCVWD Penitencia Creek Landslide



Notes: 1) File W:\geology\penitencia creek landslide\figures\fig37.grf
2) Data collected by Neil O. Anderson & Associates, Inc.

DATE 10/5/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

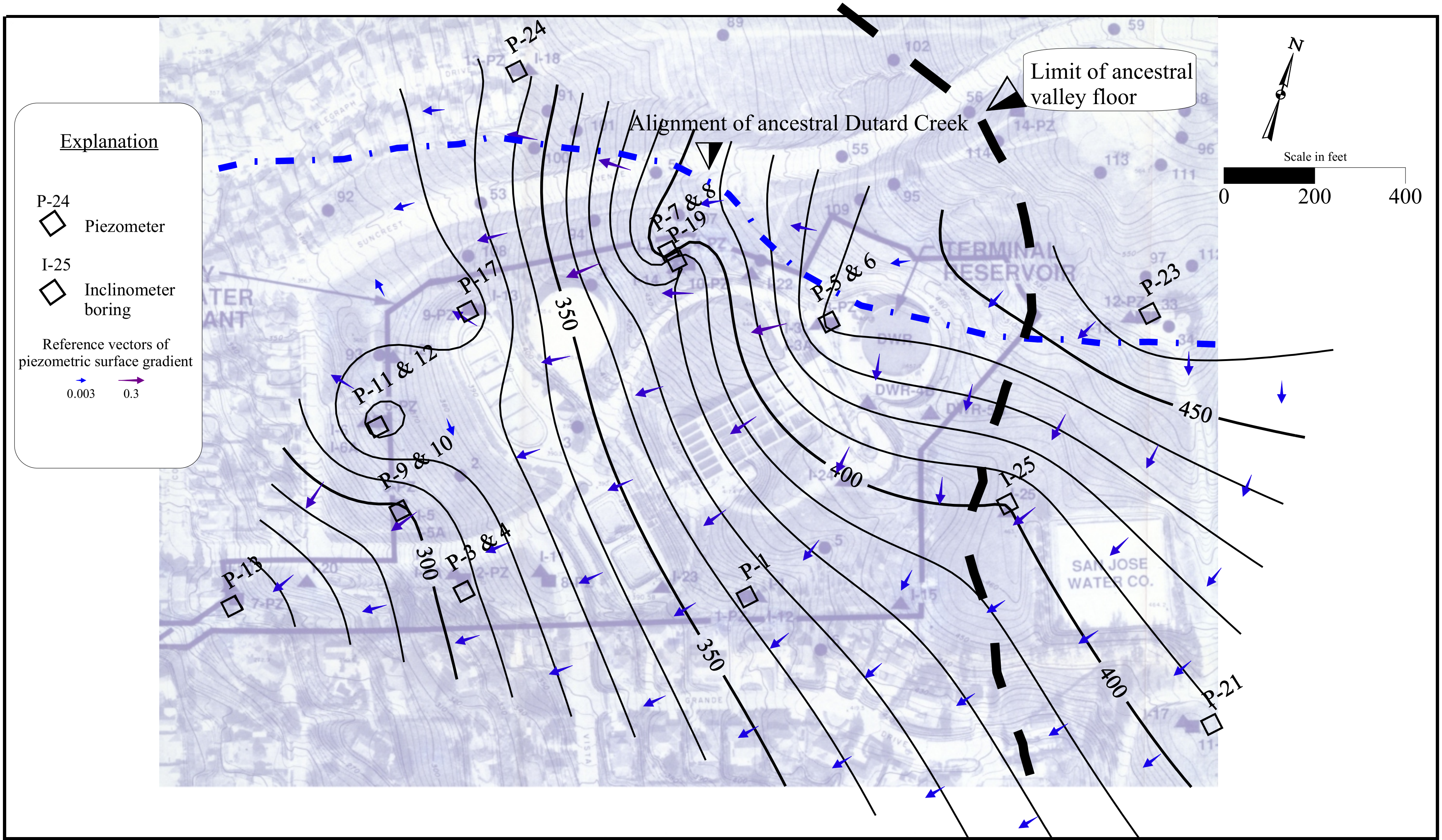
Santa Clara Valley Water District

TDR Cable Readout Plots

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
37



Note: 1) Piezometric surface contours are interpreted from limited number of piezometers and inclinometer borings shown
2) Contours were mathematically interpreted using SURFER
3) Contours are in feet based on 1929 NGVD
4) File W:/geology/penitencia creek landslide/figures/fig38.srf

DATE 1/11/01
DESIGN JLN
DRAWN JLN
CHECKED RLV

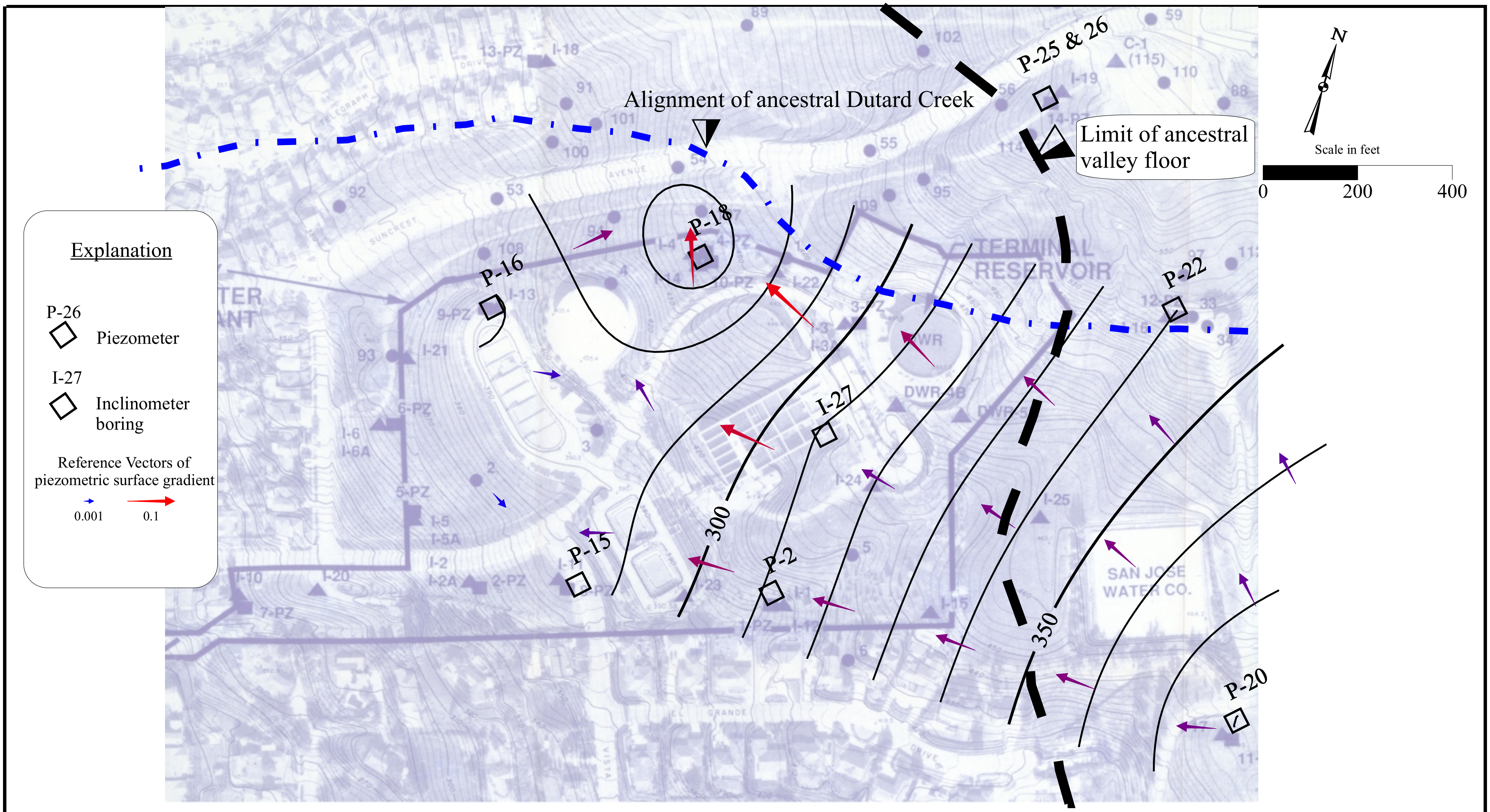
Santa Clara Valley Water District

Piezometric Surface Contour and Gradient Map of Shallowest Piezometric Surface September 1999

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1" = 200'

FIGURE 38



Note: 1) Piezometric surface contours are interpreted from limited number of piezometers and inclinometer borings shown
2) Contours were mathematically interpreted using SURFER
3) Contours are in feet based on 1929 NGVD
4) File W:/geology/penitencia creek landslide/figures/fig39.srf

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DRAWN	JLM
CHECKED	RLV

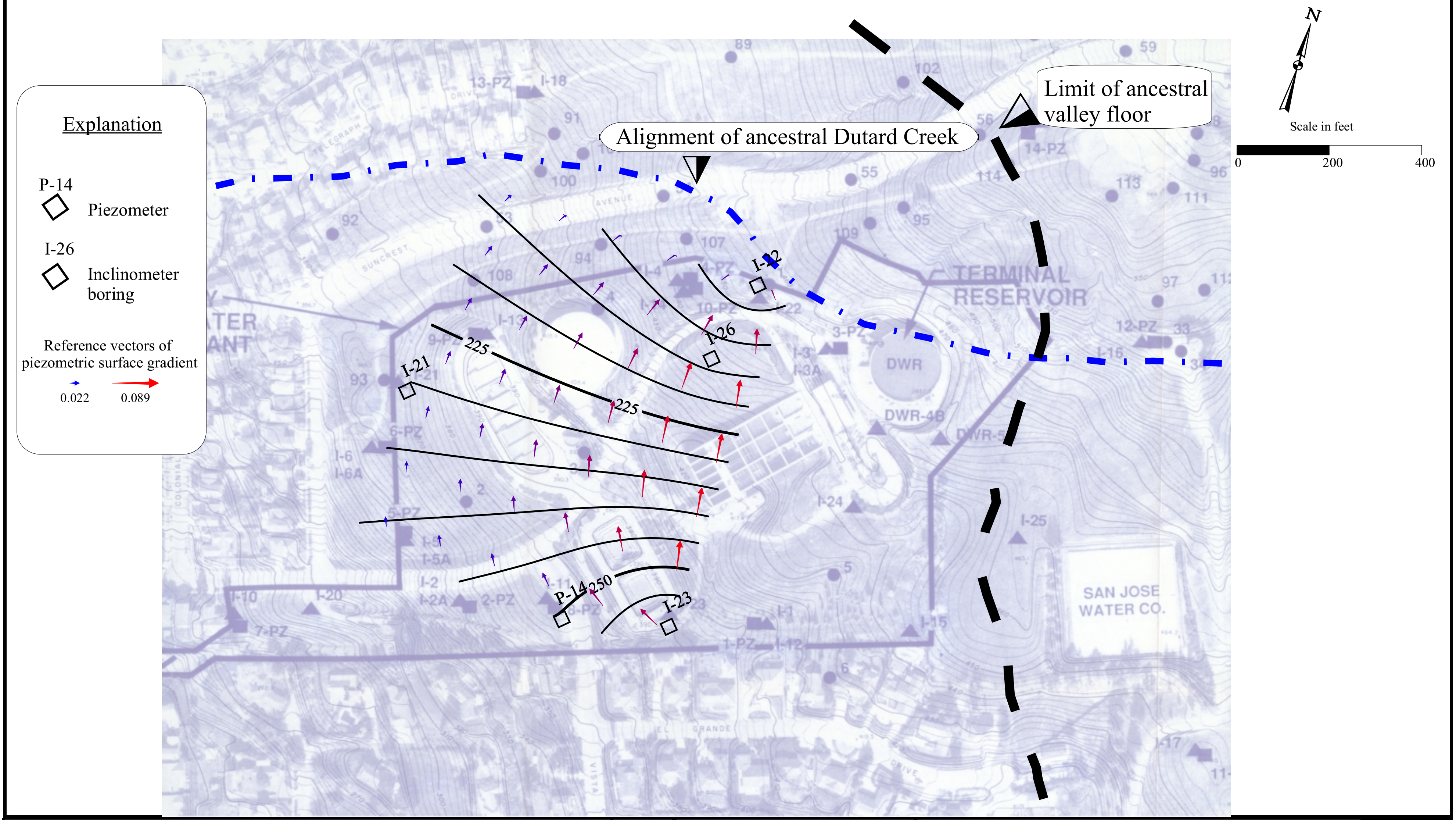
Santa Clara Valley Water District

Piezometric Surface Contour Map of Landslide Piezometric Surface September 1999

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1" = 200'

FIGURE
39



Note: 1) Piezometric surface contours are interpreted from limited number of piezometers and inclinometer borings shown
2) Contours were mathematically interpreted using SURFER
3) Contours are in feet based on 1929 NGVD
4) File W:/geology/penitencia creek landslide/figures/fig40.srf

DATE	10/5/00
DESIGN	JLM
DRAWN	JLM
CHECKED	RLV

Santa Clara Valley Water District

Piezometric Surface Contour and Gradient Map of Deepest Piezometric Surface, September 1999

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1" = 200'

FIGURE
40

7.0 DISCUSSION ON LANDSLIDE MECHANISM

This chapter provides a discussion of the factors that likely contributed to development of the Penitencia Creek Landslide. As discussed in Chapter 3, the landslide has developed and formed within the Santa Clara Formation, which is prone to landsliding on sloping ground along the eastern and western margins that flank the mountains of the southern San Francisco Bay Area. Since the Penitencia Creek Landslide was originally identified in the technical literature by Nilsen and Brabb (1972) (which coincidentally was the year that the District began its surveillance program to monitor suspected movements of the landslide) numerous other similar landslides have been noted within the Santa Clara Formation at other locations in the southern Bay Area. By the early 1980's, researchers were beginning to investigate the unique shear strength characteristics of the Santa Clara Formation. Nelson (1985, 1992) attributed massive landsliding in the Santa Clara Formation to progressive shear failure within brittle claystone units. The primary geologic factors controlling landsliding in the Santa Clara Formation claystone are the presence of a smectite clay mineral and the overconsolidated nature of the claystone (Nelson 1992).

The Santa Clara Formation claystone materials, because of their overconsolidation, typically possess peak shear strengths greater than soil, but significantly less than rock. Due to their brittle nature claystone rapidly loses strength as it weathers. This rapid strain softening, combined with the presence of smectite clay minerals, results in the development of low residual friction angles along landslide failure surfaces. These concepts are further explained in the following sections.

The general geologic constraints that lead to the development of the Penitencia Creek Landslide include the following; 1) deposition of lacustrine (clayey) sediments, 2) subsequent overconsolidation of these sediments to form claystone, and 3) removal of overburden to initiate progressive failure. The inter-relationships of these mechanisms are discussed below in further detail.

7.1 Landslide Geometry

The maximum measured thickness of the landslide beneath the PWTP is about 270 feet. Cross section analysis of inclinometer data indicates that the basal shear surface beneath much of the landslide is nearly horizontal (Figures 6 through 10). Resultant vector data from inclinometer borings shows the direction of displacement of the central portion of Penitencia Creek Landslide in the vicinity of the PWTP to be generally southwestward with azimuth bearings ranging about 192 degrees to 239 degrees (quadrant bearings of S12^{O}W to S59^{O}W). Vector data from GPS monument line surveys, however, show different displacement directions depending on location of the monument line. However, there may be a question regarding the stability of the GPS monitoring stations, which consist of reflective washers ("shiners") nailed into the street pavement.

7.2 Residual Shear Strength

When dealing with overconsolidated clay materials similar to those of the Santa Clara Formation, it is important to differentiate between two possible conditions of shear strength when evaluating slope stability issues. These two conditions are customarily referred to as the "peak" and "residual" shear strength and are due entirely to processes associated with clay mineralogy and geologic history. Peak shear strength is indicative of the maximum shear strength of a soil mass. The peak shear strength governs the stability of slopes that have not experienced slope failure. It is a measure of the maximum resistance to a shear force and is dependent on the strength of the bonds formed within the clay minerals due to a combination of consolidation and/or cementation. On the other hand, for slopes that have experienced failure, the residual shear strength appears to govern the shear strength along the failure surface. Residual shear strength of a clayey soil is defined as the minimum strength resistance, and is entirely dependent on the type of the clay minerals present. Once the residual shear strength has been developed, it is typically characterized by the development of a unique shear surface that is referred to as a "slickenside" surface, defined as a polished, striated competent surface caused by one rock mass sliding over another along a planar surface. In clay-rich bedrock units, similar to the Santa Clara Formation, the slickenside surface is characterized by a highly polished surface (i.e., similar to a waxed tabletop) that develops as a result of preferential alignment of clay particles during landslide movement. Clay particles have a shape like that of a playing card. In the case of peak shear strength, the clay particles are bonded in an edge-to-face alignment (i.e., flocculated structure) in a manner consistent with and dictated by the chemistry of the preferentially bonded water (termed diffuse double-layer). Development of residual shear strength begins with deformation of the randomly bonded clay particles. These bonds are continuously deformed as a result of slope movement until, when the ultimate residual (frictional) strength is developed, the clay particles are aligned parallel to the failure surface. This is the weakest possible configuration. It has been estimated (Kanji, 1974) that about 2-3 feet of shear displacement is necessary for the shear strength to degrade from the peak-to-residual value in natural slopes.

In one of the most important discussions on residual shear strength in the technical literature Skempton (Skempton, 1964) reviewed our knowledge on residual strength, and compared it with values of the shear strength computed from landslides in overconsolidated plastic clays. In his paper Skempton drew three important conclusions:

1. The residual shear strength parameter fr (residual friction angle) is independent of the original strength of the clay and such factors as water content and liquidity index. The value of fr of a particular clay fraction seems to depend only on the size, shape and mineralogical composition of the clay constituent.

2. The average shear stress along the failure surface, computed from a number of landslides in overconsolidated plastic clays, bears much more resemblance to the residual shear strength than to the peak strength, and an analysis of some landslides in natural slopes showed that the shear stresses at failure were very nearly equal to the residual shear strength.

3. As a consequence of this finding, Skempton concluded that the landslides in overconsolidated clays were preceded by a progressive development of a failure surface. In natural slopes, where sufficient time has been available for the development of sliding surfaces by progressive failure, the ultimate stability depends on the residual shear strength only.

Results of shear strength studies by Kenny (1976) and Olson (1974) on commonly occurring clay minerals such as kaolinite, illite, and smectite (montmorillonite) indicate that smectite clay minerals possess the lowest residual shear strength values. Kenny (1967) determined that sodium smectite possesses residual shear strength values, expressed as tan f'_r (tangent of the residual friction angle), of 0.2 or less (i.e., less than about 11 degrees). R.L. Volpe & Associates (1987) performed X-ray diffraction and electron microscopy of the clay fraction (less than 2 mm) of soils from the shear surface of Penitencia Creek Landslide, and found that smectite clay minerals are predominant in the Santa Clara Formation. Results of X-ray diffraction analysis by Nelson (1992) on Santa Clara Formation claystone samples from the Saratoga Foothills, California, confirm that smectite and kaolinite are the two most common clay minerals, with an average ratio of smectite to kaolinite of approximately 1.8 to 1. Results of residual shear strength testing by Nelson (1992) indicate that seven of fourteen claystone samples from the Saratoga Foothills posses tan f'_r values ranging from 0.08 (4.6 degrees) to 0.149 (8.5 degrees); a strong indication that sodium smectite-rich clay minerals are present in Santa Clara Formation claystones. As indicated in Chapter 4, the residual friction angle of the Santa Clara Formation materials from the Penitencia Creek Landslide, based on 24 separate samples from throughout the landslide, ranges between 7.5° degrees to 12.2° based on the range in the Plasticity Index (Kanji, 1974).

7.3 Progressive Shear Failure as Landslide Mechanism

In describing the concept of progressive shear failure it is necessary to differentiate between the vertical and horizontal stresses acting within normally consolidated and overconsolidated clay materials. For purposes of this discussion, a normally consolidated material is defined as one that has never had a greater vertical stress than the current vertical stress. Conversely, overconsolidated clay has an existing vertical stress less than the maximum vertical stress that was developed at some time in the geologic past. For clayey soils, the relationship between the current state of stress and the maximum past pressure is customarily determined by performing a one-dimensional consolidation test in the laboratory. It is convenient to describe the degree of overconsolidation by comparing the ratio of the estimated maximum past vertical pressure to the existing vertical pressure. This ratio is referred to as the overconsolidation ratio (OCR). The correlation between vertical and horizontal stress for normally consolidated (N.C.) material is as follows:

$$'_h = k_o * \ '_v$$

where:

$'_h$ = horizontal effective stress

k_o = at rest pressure coefficient » 2/3 for N.C. clay with PI = 40

$'_v$ = vertical effective stress

Thus, for normally consolidated clay material with a Plasticity Index equal to 40 the horizontal stress is about equal to 2/3 of the vertical stress. In the case of a heavily overconsolidated clay, on the other hand, the horizontal stress can be between 1.5 (OCR=8) and 2.5 (OCR=32) times the vertical stress, depending on the OCR (Lambe & Whitman, 1969). This means that the horizontal stress for a heavily overconsolidated clay is between 2.25 and 3.75 times greater than that for normally consolidated clays. The effect of this anomalous stress condition is such that it provides an understanding of the development of progressive failure and why, for example, for heavily overconsolidated clays that the gravity force does not contribute to the development of shear strength.

Bjerrum (1966) developed the concept of "progressive failure" as a mechanism for initiating landslides in natural slopes comprised of heavily overconsolidated clayey materials often referred to as claystone or clay shale. During initial consolidation, as discussed earlier in this section, the consolidation process imparts an applied energy that causes a strong bonding between clay minerals. Bjerrum referred to this energy as "recoverable strain energy." The amount of recoverable strain energy stored increases as plasticity and the amount of active clay minerals in the claystone increase.

For slopes comprised of overconsolidated claystone, there is a high potential for progressive failure if the geometry of a slope is such that large horizontal stresses run generally parallel to the slope, as opposed to other possible orientations. These large stresses can develop in the following manner. As discussed earlier in this section, after a claystone mass has experienced considerable consolidation from overburden pressures, vertical stresses become greater than horizontal stresses. This situation gets reversed when the overburden is removed. With subsequent erosion of overburden the overconsolidated claystone material tends to expand toward ground surface. The tendency for lateral expansion, however, is inhibited by confinement from surrounding stratum. Consequently, very large horizontal stresses will develop relative to vertical stresses. Localized failure is then possible because the greatest stress will generally be parallel to the slope. Also, it should be noted that overconsolidated claystones tend to be brittle because of the presence of strong bonding between clay minerals induced by the overconsolidation. Therefore, only small amounts of strain are required to reduce large horizontal stress in brittle materials.

According to Bjerrum (1966), several requisite conditions are necessary for the development of progressive failure in slopes of overconsolidated claystone. Progressive failure results in the development of a continuous landslide shear surface. To initiate progressive failure, a discontinuity must be present somewhere in the claystone mass, or at a contact between different lithologies. The discontinuity becomes the focal point for development of concentrated shear stresses and subsequent deformation. Examples of different types of discontinuities include soft spots, changes in material characteristics, a cut at the toe of a slope, etc. Progressive failure will begin at the discontinuity and, depending on the location of the discontinuity, proceed both upslope and downslope as local concentrations of shear stress exceed the peak shear strength of the claystone mass. The failure surface advances when differential strains along the zone of failure are great enough to strain the claystone mass beyond the peak strength. The claystone mass must then experience a large and rapid decrease in shear strength with small amounts of strain once the shear strength has been exceeded. This reduction in shear strength is known as rapid strain softening, which involves breaking down the strong bonds developed between the clay minerals during consolidation. Without rapid strain softening, there would be resistance to shear along the failure zone and any additional movement would be inhibited. This is why overconsolidated claystone masses in particular are prone to landsliding. Finally, concentrated stresses that initiated failure must be allowed to proceed into unsheared portions of the claystone mass.

Once the process of progressive failure is complete a continuous shear surface has developed that will tend to parallel the existing ground surface, or follow the direction of bedding. With time, massive landsliding will occur because resistance along the landslide surface will not be able to counteract the downslope shearing stresses imposed on the landslide surface by the weight of the landslide mass above it. Sufficient movement of the sliding soil mass will reduce the shear strength along the shear surface from the peak strength to a value close to or at the residual shear strength.

7.4 Proposed Landslide Model for Penitencia Creek Landslide

This section provides a proposed landslide model that is based on the Bjerrum concept of progressive failure described in Section 7.3 and the unique physical characteristics of Penitencia Creek landslide. As discussed in Section 3.6, we estimate that the landslide probably initiated some 18,000 to 20,000 years ago, and still remains active. Our model attempts to address the following questions: 1) What initiated landslide movement? 2) How was a relatively horizontal shear surface formed at the base of the landslide mass? 3) What mechanisms provide relatively continuous ongoing landslide creep along a horizontal shear surface? 4) How does the lobate-shape of the landslide toe as it encroaches on the valley floor relate to inclinometer and survey line movement vectors, and the right-angle deflection of Dutard Creek? 5) How much longer will the landslide remain active?

To aid the discussion, we have divided our landslide model into six stages as depicted in Figure 41. A general discussion of slope evolution involving the Santa Clara Formation just prior to landslide initiation and after full development of the landslide is presented in Chapter 3. The following sections provide a specific discussion of the intervening steps in landslide evolution, presented as cross section schematic drawings 1 through 6 in Figure 41.

7.4.1 Landslide Evolution

Figure 42 is an interpretive contour map of the basal or main landslide shear surface of Penitencia Creek Landslide. This map consists of lines of equal basal landslide (shear) surface elevations (using Surfer software) based on subsurface data from inclinometers and borings. We believe that portions of the contour map represent ancestral slope conditions prior to landslide development. The landslide mass probably formed on the uppermost steeper slope contours and moved onto and enveloped the lowermost flatter slope contours. This interpretation is based on discordant relationships between various characteristics of the landslide and the main shear surface, and the relative position of pertinent physical and geologic features. The following paragraphs discuss the pertinent physical and geologic characteristics of ancestral conditions indicated by the basal landslide surface contours (Figure 42). Because the interpretative contour map may represent ancestral conditions, it will be used to explain various stages of our landslide evolution model.

When slope failure has progressed to the point where a continuous failure shear surface has developed, the aspect or orientation of the landslide surface is generally the same as the aspect of the underlying shear surface (i.e. the failure plane roughly parallels ground surface). For example, if the failed ground surface possesses a northwesterly aspect the underlying failed surface will also possess a northwesterly aspect. As shown in Figure 42, contours on the majority of the landslide mass have a southwesterly aspect and the landslide is moving generally southwestward. However, the main shear surface contours beneath the northern portion of the landslide have a discordant north-northwesterly aspect. Consequently, the northern portion of the landslide appears to be moving parallel to underlying shear surface contours as opposed to an expected slope-perpendicular motion.

We speculate that the relatively flatter slopes of the basal failure surface beneath most of the landslide is likely the general topography of the ancestral Santa Clara Valley, upon which the landslide mass flowed early in its movement history. Consequently, the boundary between the ancestral valley floor and the steeper eastern slopes is represented by the 300-foot contour of the basal shear surface.

The apparent ancestral location of the westernmost thrust fault (Clayton) is suitably positioned near the base of the ancestral slope. Range front thrust faults typically form near the base of slopes.

The northernmost contours of the main shear surface (Figure 42) have a “V-shape” in plan view indicative of an incised ancestral channel, with the closed end of the “V” pointing generally eastward. Topographically, “V” contours “point” in the direction that stream flow originates. In addition, these subsurface “V-shaped” contours align with present day “V-shaped” contours of the uppermost channel of Dutard Creek, above about el. 520 feet. It appears likely that the ancestral Dutard Creek produced the subsurface channel prior to initiation of Penitencia Creek Landslide. The presence of an ancestral channel is supported by subsurface data from inclinometer boring DWR-6, recently drilled near the axis of the apparent channel alignment to a depth of about 300 feet. The lower 100 feet of the boring consists predominantly of clayey sands with gravels and cobbles, which could have been deposited by an ancestral stream.

7.4.1.1 Stage 1 - ~18,000 to 20,000 years before present

As shown in Figure 41, Stage 1 represents antecedent conditions prior to the earliest development of the landslide. Oversteepened slopes (i.e. >12 degrees) would have characterized this pre-slide condition. As mentioned in Chapter 3, the Bay Area began to experience a wetter climate about 18,000 to 20,000 years ago, during the last maximum glacial period. This wetter period would have significantly raised groundwater levels.

By this time, the range front imbricate-thrust-fault system had become fully developed. The dip-slip component of movement along these faults resulted in uplift of the Diablo Range, which in turn increased slope gradients and caused rapid erosion. Uplift resulted in significant removal of overburden sediments, which included the uppermost coarse-grained debris-flow-alluvial fan deposits of the Santa Clara Formation.

7.4.1.2 Stage 2 - Begin development of progressive failure of slope seated in over-consolidated claystone

Stage 2 marks the beginning of development of large ancient landslides in the Bay Area like Penitencia Creek Landslide. The mechanism of progressive failure discussed earlier in this chapter would begin in underlying claystone units due to the combined affects of geologic processes outlined for Stage 1. These processes include 1) increase in slope gradients (oversteepened slopes), 2) removal of overburden stresses, and 2) raising of groundwater levels.

The likely minimum slope inclination required to initiate slope failure would have been about 12 degrees, based on residual shear strength values of the sodium-smectite clay minerals of the Santa Clara Formation claystone units. Slopes underlain by sodium-smectite-rich claystone will attempt to achieve inclinations less than the maximum residual friction angle, or about 12 degrees. Based on contours of the main landslide shear surface, (Figure 42) the ancestral base of the landslide probably developed on the steeper contours to the east. These contours have a gradient of about 15 to 19 degrees. Because progressive failure usually follows the surface slope, it appears likely that the ancestral surface slope may also have been about 15 to 19 degrees prior to failure.

After removal of overburden pressures, the underlying claystone units would have begun to experience significant erosion problems due to their overconsolidation. As described earlier in this chapter, heavily overconsolidated clay shales are susceptible to rapid softening because of the existence of much higher lateral pressures (parallel to the ground surface) relative to vertical pressures (perpendicular to ground surface), which results in increased erosion. The higher groundwater levels would increase buoyancy effects and decrease the effective stress acting within the underlying claystone units. Higher groundwater levels would also cause weathering and rapid softening of the overconsolidated claystone.

Other factors that may have contributed to landslide development would include significant earthquakes and possibly rapid stream erosion at the toe of the slope. According to Bjerrum (1966), initiation of progressive failure requires a discontinuity somewhere in the slope. The location of imbricate-thrust faults near the base of the slope would form a fault-related discontinuity such as local folding and/or earthquake induced rupture. Thus, movement along the underlying Clayton, Crosley, or Berryessa faults could have been enough to initiate progressive failure.

7.4.1.3 Stage 3 – Fully developed progressive failure mode of landsliding

During Stage 3, a major section of the west-facing slope of the Diablo Range was probably involved in landsliding fairly quickly with respect to geologic time. The landslide was probably fully developed within several hundred years, if not sooner. The landslide mass probably enlarged laterally upslope increasing in size to include a significantly larger area than what remains of the landslide mass today. As the landslide formed, it consumed successive failed wedges of slope material. The first failed wedge probably was relatively localized and formed near the base of the slope. Minor downslope movement of the first wedge would result in removal of lateral support from intact (unfailed) upslope portions. With no lateral support a new failure wedge would begin to form. In this manner, successive failed wedges would form laterally uphill, thus enlarging the landslide mass. It appears likely that during each successive uphill failure of a new wedge the main shear surface would penetrate to a greater depth than the previous wedge. A deeper shear surface would coincide with enlargement of the landslide mass and would develop as deeper portions of the overconsolidated claystone began to experience rapid softening due to deeper weathering. This mechanism would explain the presence of multiple shear surfaces noted on logs of previous inclinometer borings (DWR, 2000).

7.4.1.4 Stage 4 - Continued Expansion of Landslide Mass and Erosion Begins

Stage 4 marks the time period when the landslide mass has reached its extent laterally and vertically. From this period to the present, erosion and translation of the landslide mass has become the predominant ground surface slope process. Erosion would have begun to alter the land shape. Older landslide headscarps would become more rounded and subdued due to attack by overland flow of rainfall runoff. Based on previous mapping, we estimate that the original slide may have propagated up to the Hayward fault, about 1.4 miles upslope from the existing PWTP, since Coyle (1984) mapped a relatively isolated segment of Santa Clara Formation (see Figure 14) along the western boundary of the Hayward fault.

7.4.1.5 Stage 5 - Begin Encroachment of Landslide onto Valley Floor

This step is necessary because of the need to lower the overall average slope of the failed landslide surface. As the landslide encroached out onto the ancestral valley floor, the overall slope of the failed mass lessened, as did the overall driving force. The landslide mass probably stopped expanding several thousand years ago. Since that time the areal extent of the landslide has been decreasing due to erosion. The upper portion of the landslide has been completely lost due to erosion with the current exposed slope above the Berryessa fault zone comprised of Berryessa Formation. Two separate events were necessary to either stop or slow down the landslide, which can be considered natural landslide mitigation measures. One is the erosion of the upper portion of the landslide to decrease driving forces (i.e., removal of the ancestral Santa Clara Formation). The other event began when the landslide encroached onto the valley floor. The relatively low residual shear stress along the slide surface may have begun to increase slightly (causing the landslide rate to slow down) as it encountered the more granular soils of the valley floor. Completion of the first event, has already occurred, or nearly so. The second, which includes the lateral translation process, is still occurring as more of the landslide shear surface encounters resistance imparted by the valley floor sediments. Based on the orientation of the failure surface, it is estimated that the landslide may have moved as much as 1500-2000 feet out onto the valley floor. The entire duration of this schematic is intended to accommodate enough time to allow continued downslope movement until the overall surface slope of the entire sliding mass reaches a slope angle of less than 12 degrees, the maximum residual angle of the smectite-rich Santa Clara Formation.

7.4.1.6 Stage 6 - Existing Penitencia Creek Landslide

As shown in Figure 41, the current landslide represents only a relatively small portion of what was the original ancestral landslide mass. There does not appear to be any potential for further upslope enlargement of the landslide mass. The headscarp of the original landslide has been eroded away. Surface slopes have nearly reached minimum residual friction angles of the Santa Clara Formation (~6 to 8 degrees). Instead of constant rates of movement, which probably existed for much of the life of the landslide, a rise in groundwater level is now required to reactivate movements. Although still considered active, the average rate of landslide has dropped from initially relatively high rates to about 0.3 to 0.5 in/yr. over the majority of the landslide. The landslide is capable of continued active landsliding, at successively slower rates of movement, until such time that the overall average slope of the landslide surface has reached an inclination of less than 6 to 8 degrees, the estimated average residual friction angle of the underlying Santa Clara Formation. We are reasonably certain that the Penitencia Creek Landslide will remain active for the anticipated life of the PWTP facility, and may continue for several hundreds to thousands of years.

7.5 Rates of Landslide Movement

As discussed in Section 7.4, the Penitencia Creek Landslide has been active for an estimated 20,000 years. During that protracted period, the rate of landslide movement has probably varied considerably. Landslides are known to have a significant range in rates of movement. For example, landslides that involve catastrophic rates, such as debris flows or quick clays, have been estimated to move at many tens of miles per hour. For other types of landslides, movement rates will typically decay with time since the landslide mass moves toward a more stable condition. In order to capture this broad range over which landslides move, Turner and Schuster (TRB,1996) have developed a new landslide velocity scale that is reproduced as Table 4. Their scale is an update of an older landslide movement scale originally developed in 1978 by Varnes (1978).

As shown on Table 4, the rates of movement cover several orders-of-magnitude, making it necessary to define the broad range of landslide movement by a logarithmic scale. The Penitencia Creek Landslide is currently moving at an average rate of between 0.3 to 0.5 in/yr. According to Table 2, this rate of movement correlates to a Velocity Class 1, *Extremely Slow,* the lowest velocity classification, which has an upper bound rate of 16 mm/yr. (0.62 in/yr.).

Table 4 - Landslide Velocity Scale by Turner and Schuster

Velocity Class	Description	Velocity (mm/sec)	Typical Velocity
7	Extremely Rapid	5×10^{3}	5 m/sec
6	Very Rapid	5×10^{1}	3 m/min
5	Rapid	5×10^{-1}	1.8 m/hr
4	Moderate	5×10^{-3}	13 m/mo
3	Slow	5×10^{-5}	1.6 m/yr
2	Very Slow	5×10^{-7}	16 mm/yr
1	Extremely Slow	$<5 \times 10^{-7}$	< 16mm/yr

As stated earlier, we estimate that the landslide has moved about 1500-2000 feet onto the valley floor over its lifetime. Assuming that this total movement occurred at the same constant rate of 0.5 in/yr., a period of between 36,000 and 48,000 years would be required to account for the estimated total movement. As mentioned in Section 7.4, the landslide is estimated to have begun about 18,000 to 20,000 years ago. It appears likely the average rate of movement for the landslide has been considerably faster than what is currently being measured. Consequently, we estimate that, over the life of the landslide, the rates of movement may have been as high as Velocity Class 3, *Slow*, with annual magnitudes of movement of perhaps 5-10 ft/yr.

In assessing the overall movement of the landslide one must also consider historical earthquakes that must have impacted the site. For example, in reviewing Table 1, it is noted that the recurrence intervals for characteristic earthquakes along the San Andreas fault (M=7.9) and the Hayward fault are about 210 years and 220 years, respectively. These are the two major fault systems to impact the site. This suggests that between 80 and 95 such events would likely have impacted the site during the 18,000-20,000 lifetime of the Penitencia Creek Landslide. It is not clear how much of the total landslide movement has been earthquake-induced, but it probably has been significant.

7.6 Physiographic Features Related to Landsliding

In the following paragraphs we discuss physiographic (physical-geographic) features related to the Penitencia Creek Landslide that do not fit specifically within a particular evolution stage, but do fit within the context of our proposed landslide model.

7.6.1 Serpentinite Masses Relative to Secondary Creep Zones

Past District geologists and consultants have speculated that secondary creep zones detected in inclinometer borings may be seated within relatively shallow serpentinite masses encountered in some borings beneath the PWTP. There has also been speculation that the secondary creep zones (and supposedly associated serpentinite masses) could represent dormant shallow landsliding. The concern is that dormant shallow landslides have the potential to be reactivated by making major cuts or by adding weight with construction of heavy structures and large fill prisms.

To evaluate the above-mentioned concern, we developed cross sections showing depth intervals of serpentinite masses from boring logs and zones of creep from inclinometers. The findings of our cross section analysis indicate that the relative geometry of the serpentinite and creep zones is evident as a fairly random distribution. The elevations of the serpentinite masses do not "line up" with the individual creep zones from one boring to the next. In fact, the creep zones do not "line up" from one boring to the next, as would be expected if they represented a previous shallow landslide mass that is now dormant. We believe that the serpentinite masses are randomly distributed within the Penitencia Creek Landslide mass and were probably incorporated into the mass as the landslide progressed downhill, or represent materials that were originally in the Santa Clara Formation. The creep zones represent isolated secondary adjustments of the overall landslide mass due to shrink-swell of the underlying clayey mass as it goes through successive cycles of being relatively wet or relatively dry.

Based on our analyses for this report, we believe that the likelihood of shallow landsliding beneath the PWTP is remote. As stated elsewhere, the ground surface slope gradient at the site is at, or near, the likely value of the residual friction angle for the underlying Santa Clara Formation claystone material. Consequently, there is no significant driving force to initiate movement of a shallow landslide. Some geologists and geotechnical engineers may think the presence of the active 3-acre landslide directly upslope of the Terminal Reservoir would support the argument that shallow landsliding is possible beneath the PWTP. However, that landslide was activated due its occurrence on a natural slope that is inclined slightly greater than the residual friction angle of the underlying claystone material. Another contributing factor to the activity of the landsliding is the ongoing removal of the toe by DWR as it performs routine maintenance of removing landslide debris from around the Terminal Reservoir.

In addition, we believe that shallow landsliding will not likely be initiated with the addition of weight to the mass from the construction of heavy buildings or large fill prisms, because they would not contribute a significant driving force. We do have some concern that major cuts (greater than 25 to 35 feet high) could cause localized ground movements. Such cuts would require suitable shoring and close monitoring for significant amounts of movement.

7.6.2 Lobate-Shape of Landslide Toe and Related Movement Vectors

Historic aerial photographs taken in 1950 and 1954 show the landslide mass prior to human encroachment (see Appendix D). Along its toe, the Penitencia Creek Landslide (and other related ancient landslides to the north) possesses a distinct lobate form (lobe-shape) that is analogous to the shape of a glacier as it flows from mountainous terrain onto a flatter valley floor. The lobate shape of the landslide does in fact appear to be due to the slow-creeping flow that the landslide has been experiencing as it moves onto the valley floor during the last several thousand years or more.

The overall general shape of the landslide toe can be divided into two distinct lobes; a northwestern lobe upon which the PWTP lies, and a southeastern lobe that includes all of the landslide mass southeast of the PWTP as shown in Figure 42. Four of the six-monument line surveys cross the landslide toe. Of these four, monument survey lines 2 and 3 cross the southeastern lobe and monument survey lines 4 and 5 cross the northwestern lobe. Results of the monument survey line data from lines 2 and 3 indicates that the southeastern lobe may be creeping south-southeastward, which is roughly 90 degrees from the predominant southwestward motion that most of the other monitoring points on the landslide mass are experiencing. This 90-degree sense of movement appears to be producing the shape of the southeastern lobe and reflects a similar sense of deflection that Dutard Creek has historically experienced (described in the following section).

7.6.3 Right Angle Deflection of Dutard Creek

As stated in the previous Landslide Evolution section (7.4.1), it appears likely that the ancestral Dutard Creek produced a generally southwest-trending subsurface channel prior to initiation of Penitencia Creek Landslide. Subsurface evidence indicates that the ancestral channel may lie beneath the northern portion of the landslide (Figure 42). The existing Dutard Creek Channel trends southwestward along its headwater in the steeper upper portion of the drainage. However, the channel is deflected almost 90 degrees to the south-southwest at about the midpoint of the landslide. The deflected channel appears to coincide with a change in the ancestral topography from steeper upper slopes to the flatter ancestral valley floor as shown in Figure 42. The deflected creek channel coincides with a trace of the Clayton fault, as mapped by Coyle (1984). While it is not certain when the creek was deflected, it appears likely that landslide movement, combined possibly with thrusting along Clayton fault, has produced the deflected creek channel. We hypothesize that as the landslide moved downslope and began to encroach onto the ancestral valley floor, the mass apparently deflected the stream flow of the ancestral Dutard Creek to the south toward Penitencia Creek. Uplift along Clayton fault would result in minor downwarping (or upwarping) of the ground surface, which would also allow preferential stream flow to the south. At present, the deflected portion of Dutard Creek appears to be riding on the southeastern lobe of the landslide mass.

7.6.4 Landslide Toe Terrace

Another noteworthy feature is what we have termed a “landslide toe terrace.” This feature consists of low-lying sloping ground that borders the active toe of Penitencia Creek Landslide along its contact with Santa Clara Valley. Although topographically this terrace feature appears to be part of the landslide toe according to the results of the street survey, the toe terrace is not actively creeping with the landslide. The toe terrace is a subtle break in slope evident on topographic maps and in the field as a minor step or terrace. When one passes onto the landslide mass from the valley floor there are two noticeable steps in topography before actually moving onto the landslide mass. The first and lowermost step is the landslide toe terrace, while the second uppermost step is the actual landslide toe. Based on monument line survey data from lines 2, 3, 4, and 5 (which cross the toe), the toe terrace does not exhibit the same magnitude or direction of movement as that of the actual landslide toe. As can be seen in Figure 42, the line monuments that possess distinct vector movement (arrows) are clearly on the second uppermost minor step, while line monuments that do not move are clearly on the first lowermost minor step. While performing aerial photo-interpretative mapping for the 1:100,000 scale San Jose sheet, Carl Wentworth (2001) observed a similar feature along the parameter of a large landslide on the west side of Santa Clara Valley near the northern end of Coyote reservoir in the southern Bay Area. He attributed this feature to uplift of the younger valley floor sediments along an outlying failure surface produced by the advancing landslide toe.

It is not clear how the landslide toe terrace developed. There may be several possible explanations. The most obvious is that the terrace has developed into a pressure ridge that has formed by the downward force of the landslide toe as it encroaches onto the valley floor. This would be analogous to the landslide toe acting like the track of a Caterpillar tractor that forms a terrace by pushing up the soft valley sediments as the landslide moves onto the valley floor. This model is discussed in more detail below in section 7.7.3.5. Another possibility is that the toe terrace is material that has eroded off of the landslide mass and onto the valley floor beyond the toe of the landslide. In this model, the landslide is moving so slowly that the processes of erosion and deposition are occurring beyond the landslide toe at a rate slightly greater than landslide movement.

7.6.5 Groundwater Anomaly at the Site

As mentioned in Chapter 3 and Chapter 7, the landslide is underlain by clayey sands and gravels of older alluvium that appear to be more permeable than the overlying highly plastic clays of the landslide mass. On the north side of the landslide, we hypothesize that the ancestral Dutard Creek is buried under the landslide. The presence of substantially more permeable material appears to be responsible for a deeper and apparently anomalous groundwater surface beneath the landslide. mentioned in an earlier section Although the piezometric surface within the landslide mass is typical for a sloping hillside (e.g., sloping piezometric surface toward the base of the slope). a deeper and totally different piezometric surface is present below the landslide surface. . This is clearly seen when reviewing the piezometric response from several of the piezometers that penetrated the landslide surface shown in Figures B-3 through B-16 in Appendix B. Contours of equal piezometric head for the lower piezometric surface are presented in Figure 40. It should be noted that due to the lack of deep piezometric data below the landslide plane east of the PWTP, the piezometric contours couldn't be extended toward the eastern side of the landslide mass. Having such additional data would be interesting. For example, as shown in Figure 42, the ancestral valley floor is shown to run in a northwestward orientation just west of the buried trace of the Clayton fault and several hundred feet uphill of the Terminal Reservoir. This suggests that east of the contact between the landslide and the ancient valley floor the material underlying landslide is the Berryessa Formation, although drill holes have not been advanced deep enough by the District or DWR to confirm this.

A review of the contours of equal piezometric head for the landslide mass (Figure 38) and the underlying pervious material (Figure 40), indicates that completely different piezometric conditions are currently acting within these two areas. The upper piezometric surface in the landslide has a seepage gradient to the southwest, while the older alluvium beneath the landslide plane has seepage gradient to the north toward the ancestral Dutard Creek. Without detailed stream gauge data, it is not clear whether sediments deposited by the ancestral Dutard Creek provide a hydraulic connection between surface flows higher up in the drainage area and the older alluvium beneath the landslide mass. The presence of such a connection would tend to support the observation that increased rates of landslide movement appear to be directly influenced by periods of higher rainfall.

7.7 Landslide Toe Location

Locating the toe of landslide has been inferred from several different sources including both surface and subsurface data. For purposes of this discussion, the study area has been divided into two broad areas. Figure 43 shows the northern study area of the landslide toe and Figure 44 shows the southern study area. It should be noted that these two figures overlap the two study areas in the vicinity of the Penvault structure where the three large pipelines leave the stable valley floor and come onto the landslide.

7.7.1 Northern Study Area

The northern study area includes the following study results: 1) an exploratory trench across the toe by Earth Systems Consultants (1990) north of Suncrest Avenue; 2) Santa Clara Valley Water District (SCVWD) Monument Line Survey No. 5 along Suncrest Avenue; 3) SCVWD Monument Line Survey No. 4 along Whitman Way; 4) the Penvault structure along the delivery pipeline alignment; and 5) logs of the overflow pipeline trench excavated across the inferred landslide toe by DWR (1966a) and the 60-in & 66-in pipeline trench (1966b).

7.7.2 Southern Study Area

The southern study area includes the following study results: 1) the DWR trenches mentioned above (area of overlap); 2) exploratory trenches across the toe by Edward Danehy (1980); 3) SCVWD Monument Line Survey No. 3 along Noble Avenue; and 4) SCVWD Monument Line Survey No. 2 along El Grande Drive. Figure 45 shows generalized logs of the DWR trenches, Figure 46 shows generalized logs of trenches by Danehy (1980) and Earth Systems Consultants (1990). Figure 47 shows a cross section through the Penvault and vicinity. Figures 48 through 51 include vertical data from each monument line survey (Nos. 5, 4, 3, and 2, respectively) and consist of plots vertical elevation changes for four selected dates (8/30/96, 11/7/97, 11/3/99, and 6/9/00) and a section view of initial survey elevations of monuments along the respective line survey. The survey data from the four monument lines are important for identifying the toe of the landslide based on both lateral movement of the various monuments and for identifying areas of uplift and down thrusting based on relative vertical movements.

7.7.3 Findings and Discussions

The following discussion is based on subsurface evidence for the presence of the landslide toe from exploratory trenches logged by other consultants/workers, and on surface evidence for the presence of the toe from line monument surveys performed by the District. Table 5 summarizes pertinent landslide toe data from field trench logs and plots of vertical monument line survey data. Based on both the subsurface trench data and line survey data, we estimate elevations where the landslide toe daylights ground surface at various locations along the toe of the landslide.

7.7.3.1 Subsurface Evidence of Landslide Toe

In 1990, Earth Systems Consultants (ESC) logged two trenches (T-1 and T-2) that were excavated for an investigation of a proposed residential subdivision north of Suncrest Avenue (Figure 43) that exposed the toe of the Penitencia Creek Landslide near its northernmost contact with the valley floor. Figure 45a shows a generalized representation of ESC's Trench 1. The landslide toe is evident on the logs as a slightly upthrusted "snout" of highly weathered and highly sheared clay that contains at least 2 slide surfaces, one at the bottom of the snout and another surface about 3-1/2 feet above the bottom surface. The logs depict the toe as being bounded above and below by massive silty clays. Approximately 5 feet downslope of the toe, Trench 1 encountered a roughly horizontal layer about 2 feet thick of gravelly clayey silt. This layer is between 6 and 9 feet below ground surface. The presence of the horizontal layer suggests that soils directly downslope of the toe are relatively undisturbed. As discussed below in more detail, the presence of relatively horizontal soil layers appears to contradict elevation survey data, which appears to consistently show relative uplift along portions of the toe terrace. Based on our projection of the slide surface to ground surface, the landslide toe daylights at approximate elevation 270 feet (Figure 45a, Table 5). In addition, the landslide surface is shown on the log as emerging near the base of the steepest portion of the slope (~12°) with the bottom of the toe dipping ~30° into the slope.

In 1966, DWR geologists logged a 1000-ft.-long trench excavated for the overflow pipeline (DWR, 1966a) that sends excess water from the treatment plant to SCVWD Penitencia percolation ponds and logged a trench for the 60-in and 66-in water delivery pipelines (DWR, 1966b) that enter Penitencia Water Treatment Plant property. The overflow pipeline trench log (Figure 45b) shows a contact between upslope Santa Clara Formation soils and downslope alluvial soils that is not specifically labeled as a landslide shear surface. However, we interpret the contact to be the landslide toe because of the relative geometry of older Santa Clara Fm soils overlying younger alluvial soils. Such a contact is more indicative of a landslide shear surface than a "normal" depositional contact. Based on the geologic law of superposition, the only way that an older soil can be found overlying a younger soil is by thrusting or overturning the sequence due to landsliding or tectonic forces. Similar to the previously mentioned log (ESC, 1990), DWR's trench log shows an apparently undisturbed subhorizontal soil layer described as a clayey gravel downslope of the inferred landslide toe. The clayey gravel layer was encountered about 400 feet downslope of the toe at a depth about 4 feet below ground surface. The slide surface daylights at approximate elevation 260 feet (Figure 45b, Table 5). In addition, the landslide surface is shown on the log as emerging near the base of the steepest portion of the slope (~8°) with the bottom of the toe dipping ~10° into the slope.

Table 5 – Pertinent Landslide Toe Data

Location (agency/ consultant)	**Suncrest Ave.** (Earth Systems Consult.)	**Suncrest Ave.** (SCVWD)	**Whitman Way** (SCVWD)	**~ 150' SE Whitman Way** (SCVWD)	**Noble Lane** (DWR)	**Noble Ave. & Noble Lane** (Edward Danehy)	**Noble Ave.** (SCVWD)	**El Grande Dr.** (SCVWD)
Type of Data/ Designation	Subsurface/ Trench 1	Survey/Mon .Line No. 5	Survey/Mon. Line No. 4	Pipejoint Closure/ Penvault	Subsurface/ Overflow pipeline trench	Subsurface/ Trench T-1	Survey/Mon .Line No. 3	Survey/Mon Line No. 2
Date	1990	1996-2000	1996-2000	1983	1966	1980	1996-2000	1996-2000
Approx. Toe Daylight Elevation (ft)	~272	~270	~260	~255 to 260	~258	~272	~265	~280
Distance of Toe Terrace Uplift from L.S. Toe	----	0 to ~200’	0 to ~170’	----	----	----	~90’ to ~220’	At toe
~Toe terrace Uplift rate	----	0.01 ft/yr	0.01 ft/yr	----	----	----	0.01 ft/yr	0.025 ft/yr

Notes: ---- = no data available or not pertinent to location

The delivery pipeline trench alignment passes through the Penvault structure. The log of the delivery pipeline trench by DWR (Figure 46a) shows an inferred near vertical contact between upslope Santa Clara Formation soils and downslope alluvial soils that daylights at approximate elevation 250 feet. The near vertical orientation of this contact is not suggestive of a landslide. Consequently, the delivery pipeline trench log does not provide strong evidence for locating the landslide toe. It appears highly likely that the toe of the landslide daylights near elevation 260 feet because this is the approximate elevation where the water delivery pipelines failed due to compression in 1983 and this is the approximate elevation of the Penvault.

In 1980, consulting geologist Edward A. Danehy logged 5 trenches excavated for an investigation of a proposed residential subdivision at the intersection of Noble Avenue and Noble Lane (Figure 44). Two of the trenches (T-1 and T-5) exposed the toe of Penitencia Creek Landslide near its southernmost contact with the valley floor. Figure 46b shows Danehy's Trench T-1 and his geologic cross section B-B' (Danehy, 1980). The landslide toe is evident on the logs as a wedge of landslide material depicted as alternating undulating layers of red and yellow silty clay. In cross section Danehy (B-B', Danehy, 1980) appears to define the base of the landslide as a wet red clay layer that dips at a shallow angle into the slope. As shown in Figure 46b, Danehy correlates the red clay layer that daylights at the ground surface in T-1 with a red clay layer encountered between the depths of 20 and 24 feet in a boring (designated as boring No. 2) located about 70 feet uphill of T-l. The logs depict the landslide toe as being "C" horizon soils (unconsolidated rock material) apparently derived from the Santa Clara Formation that are bounded above and below by dark brown expansive clay labeled as an "A1" horizon, or topsoil. Similar to the previously described trench logs (ESC, 1990; DWR, 1966a), downslope of the landslide toe Trench 1 encountered a roughly horizontal layer of gravelly to cobbly silty clay defined as "B2" horizon soils (zone of accumulation). This layer is between 5 and 10 feet below ground surface. This soil horizon was encountered approximately 7 to 7-1/2 feet beneath the landslide toe in trenches T-1 and T-5. The presence of the horizontal layer suggests that soils directly downslope of the toe are relatively undisturbed. The slide surface daylights at approximate elevation 272 feet (Figure 46b, Table 5). In addition, the landslide surface is shown on the log as emerging near the base of the steepest portion of the slope (~15°) with the bottom of the toe dipping ~5° to 10° into the slope.

In order to verify the consistency of the toe of slide location near the Penvault structure, we developed a partial cross section including the lower 525 ± feet through the landslide toe area (Figure 47). The cross section (A-A') is taken through inclinometers I-10 and I-20 located near the toe. As shown in Figure 47, the relatively flat landslide surface is constrained at between elevation 240 and 245 between I-10 and I-20. To be consistent with exposures of the toe discussed above, we show the landslide surface emerging near the base of the steepest portion of the slope (~18°) with the bottom of the toe dipping ~10° into the slope (see Figures 45a, 45b, and 46b). The toe of landslide is also shown to emerge just downslope of the Penvault structure at between elevation 255 and 260. This is about the lowest elevation for the toe of landslide and probably represents the maximum section (thickness) of the landslide.

7.7.3.2 Surface Evidence of Landslide Toe

Based on our earlier analysis of monument survey line survey data from District survey Line Nos. 2, 3, 4, and 5, established across the toe of the landslide, we defined the toe as a dashed line that separates those monuments that are obviously moving in a well-defined lateral direction from monuments that do not appear to be moving laterally (relatively stationary). As depicted on Figures 43 and 44, laterally moving monuments are shown by vector arrows, relatively stationary monuments are shown as crosses, and the landslide toe is shown as a heavy dashed line. Figures 48 through 51 show section views of Line Nos. 5, 4, 3, and 2 (from north to south) looking generally northward. The first four sections on each figure show changes in elevation of the monuments on four dates, an initial survey on 8/30/96, and selected subsequent surveys on 11/7/97, 11/3/99 and 6/9/00. The fifth and final section shows an exaggerated section view (vertical scale 4x horizontal scale) of initial elevations along each respective monument line. In the section view we also show our interpretation of the location and elevation where the landslide toe daylights ground surface based on lateral movement data.

Based on our analysis, the landslide toe daylights at approximate elevation 270 feet along monument survey line No.5 (Figure 48, Table 5), at approximate elevation 260 feet along line No. 4 (Figure 49, Table 5), at approximate elevation 265 feet along line No. 3 (Figure 50, Table 5), and at approximate elevation 280 feet along line No. 2 (Figure 51, Table 5). These data agree well with landslide toe elevation data from subsurface trenches previously referenced in this memorandum.

7.7.3.3 Discussion of Landslide Toe Elevation Data

The following are pertinent observations regarding landslide toe elevation data presented on Table 5. In the vicinity of the Penvault the landslide surface daylights at approximate elevation 260’ based on data from survey Line No.4, the location of compressive failure of the delivery pipelines in the vicinity of the Penvault in 1983, and an inferred landslide toe logged by DWR (1966) in the overflow pipeline trench located just south of the Penvault. A trench log by DWR along the delivery lines in the vicinity of the Penvault was inconclusive with respect to landslide toe location. North of the Penvault and Whitman Way the landslide toe daylights at higher elevations (~270 feet). Likewise, south of the Penvault the landslide toe daylights at higher elevations (~272’ to 280’). The presence of higher toe elevations along the northern and southern portions of the landslide mass relative to a more central position is consistent with normal landslide geometry as shown by cross section analysis through the landslide (see Figures 6 through 10 in Chapter 1). Cross sections that run roughly parallel to the lobate northeast- trending landslide toe along its southwestern margin show that near its mid-section the landslide reaches a maximum thickness relative to the landslide’s lateral margins to the northwest and southeast. As is characteristic of most landslides, as one moves toward the mid-section, the landslide mass reaches a maximum thickness and as one moves toward the lateral margins the mass reaches minimal thickness. Consequently, where the mass is thickest (mid-section) the toe will tend to daylight at a relatively lower elevation, while the toe will daylight at slightly higher elevations toward the lateral margins of the landslide mass. This explains why the landslide toe is about elevation 260 feet near the Penvault, but increases to greater elevations to the north and south.

7.7.3.4 Toe Terrace Uplift

We introduced the concept of “landslide toe terrace" which we identified as low-lying sloping ground that borders the active toe of Penitencia Creek Landslide along its contact with the Santa Clara Valley floor. Although topographically this terrace feature appears to be part of the landslide toe, based on street survey data the toe terrace is not actively creeping with the landslide. We speculated on two possible models for toe terrace formation: 1) that the toe terrace is the result of a pressure ridge that has developed by the downward force of the landslide toe as it encroaches onto the valley floor, and/or 2) the terrace represents material that has eroded off of the landslide mass and been deposited downslope of the toe.

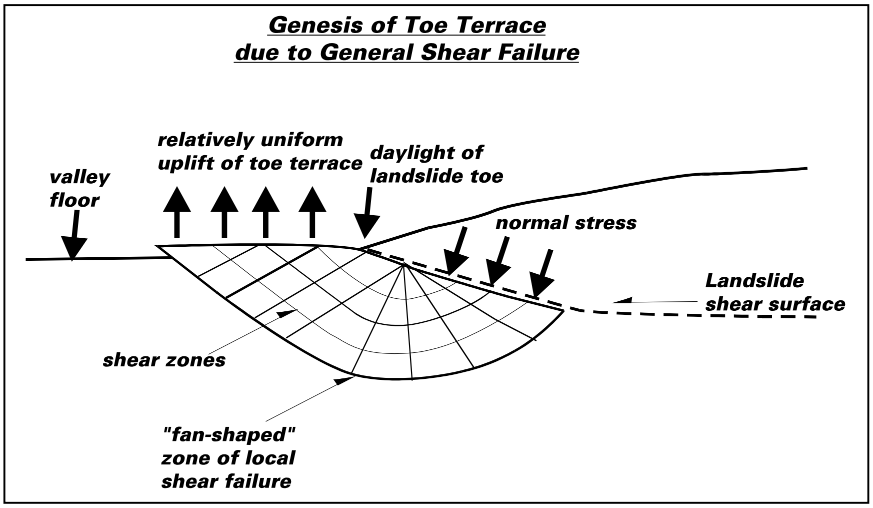
To evaluate whether the landslide toe is experiencing deformation such as relative uplift or downthrusting we plotted changes in elevation data for selected dates along survey Line Nos. 5, 4, 3, and 2, as shown in Figures 48 through 51, respectively. It should be noted that we provided a baseline for each plot that has not remained constant. For the initial survey on 8/30/96 along each survey line, the baseline is assumed to be zero (no change in elevation). Although the baseline fluctuates for subsequent surveys, the relative position of the monuments along each survey line appear to follow specific trends as discussed below.

The plots of monument elevation changes along survey line No. 5 (Figure 48) shows approximately 0.03 feet of uplift after about 4 years (~0.01 ft/yr) from the landslide toe to a distance of about 200 feet onto the toe terrace. A similar plot along survey line No. 4 (Figure 49) shows approximately the same magnitude and rate of uplift (~0.01 ft/yr) from the landslide toe to a distance of about 170 feet onto the toe terrace. Similarly, along survey line No. 3, Figure 50 shows the same approximate magnitude and rate of uplift (~0.01 ft/yr) ranging about 90 feet to 220 away from the toe on the toe terrace.

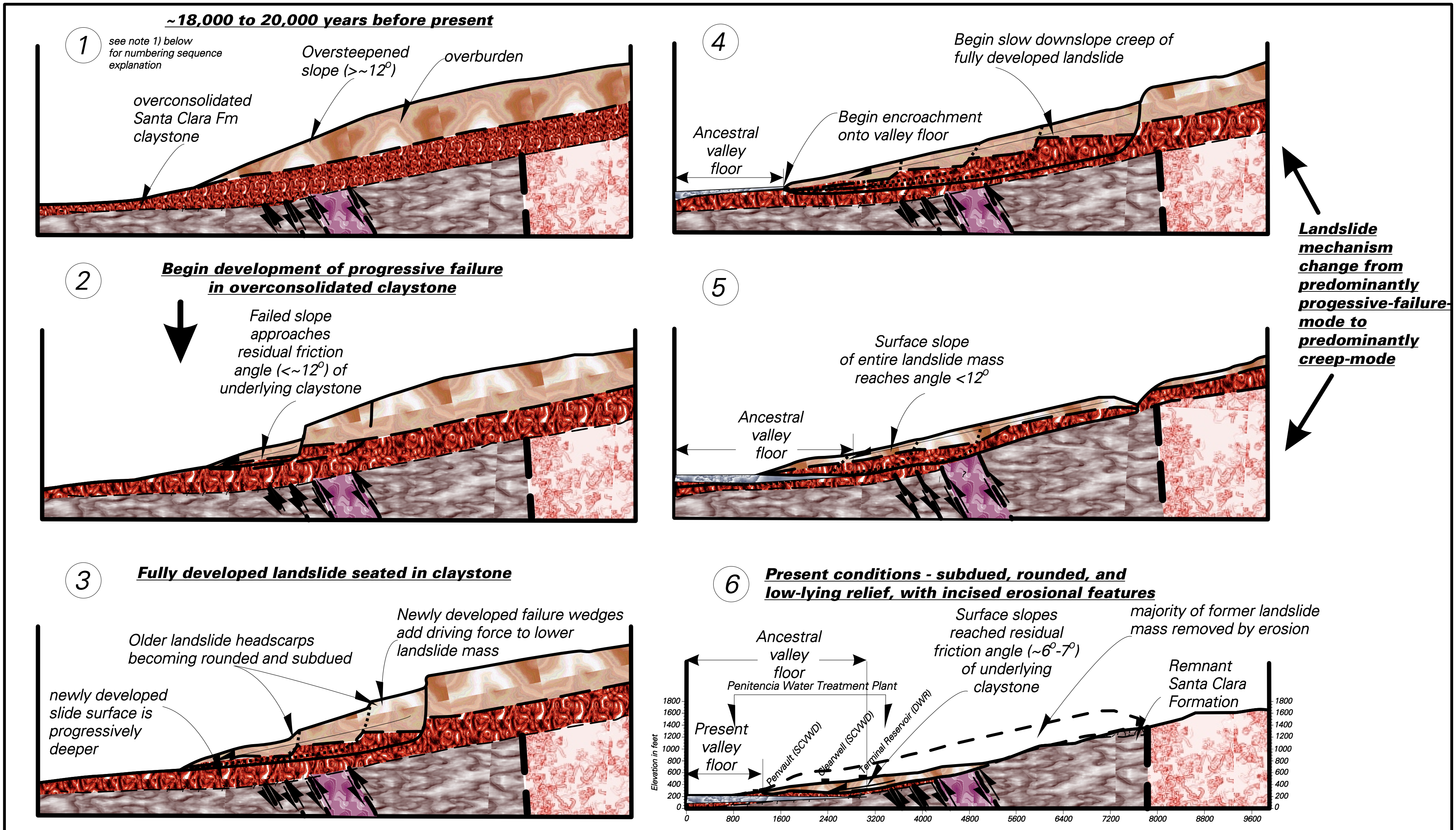
7.7.3.5 Discussion of Toe Terrace Uplift

Three trenches that cross the landslide toe (ESC, 1990; DWR, 1966; and Danehy, 1980) indicate the presence of relatively horizontal soils beneath the toe terrace directly downslope of the landslide toe. The presence of horizontal soil layers appears to contradict elevation survey data shown in Figures 48 through 50, which consistently show relative uplift along portions of the toe terrace. It appears likely that the toe terrace is experiencing a relatively uniform rate of uplift of about 0.01 ft/yr (0.12 in/yr or 3.0 mm/yr). Because relatively horizontal layers have not been otherwise deformed, the uplift appears to be uniform over several hundred feet of the toe terrace. It would appear that the relatively uniform sub-horizontal alluvial layers downstream of the landslide toe, and within at least a portion of the toe terrace, would favor the "pressure ridge" hypothesis of the presence of the toe terrace.

In a geotechnical report of landsliding along Piedmont Road for the City of Milpitas, Norfleet Consultants (1999) identified a zone of deformation downslope of a landslide toe. Norfleet Consultants describe the landslide toe as a “toe thrust” because the overall geometry is similar to a low angle thrust fault. According to the model, as the landslide mass moves downslope soils at the leading front (downslope) of the toe thrust are involved in deformation due to passive earth pressure loading, similar to that experienced at the toe of a retaining wall. A general model we developed that accounts for the toe terrace is a modification of Norfleet Consultants' (1999) model. As illustrated in the diagram below, genesis of the toe terrace is due to general shear failure. This process would be similar to bearing-capacity failure beneath a foundation. In this model a relatively rigid wedge of soil on the valley side of the landslide toe experiences passive deformation (shearing) that propagates outward until a continuous surface of failure extends to the ground surface and the surface heaves. The thrusting force at the landslide toe is equal to the driving weight of the landslide. Based on our review of trench logs by other workers across the landslide toe, it appears that this uplift may be relatively uniform.



In conclusion, the overall rate of uplift of 0.01 ft/yr for the toe terrace does not appear to be structurally significant and, most likely, would not have any major adverse effect on any new structures considered for the landslide toe area.


1
see note 1) below
for numbering sequence
explanation
~18,000 to 20,000 years before present
Oversteepened slope (>~12°)
overburden
overconsolidated Santa Clara Fm claystone
2
Begin development of progressive failure in overconsolidated claystone
Failed slope approaches residual friction angle (<~12°) of underlying claystone
3
Fully developed landslide seated in claystone
Newly developed failure wedges add driving force to lower landslide mass
Older landslide headscarps becoming rounded and subdued
newly developed slide surface is progressively deeper
4
Begin slow downslope creep of fully developed landslide
Ancestral valley floor
Begin encroachment onto valley floor
Landslide mechanism change from predominantly progessive-failure-mode to predominantly creep-mode
5
Surface slope of entire landslide mass reaches angle <12°
Ancestral valley floor
6
Present conditions - subdued, rounded, and low-lying relief, with incised erosional features
Ancestral valley floor
Penitencia Water Treatment Plant
Surface slopes reached residual friction angle (~6°-7°) of underlying claystone
majority of former landslide mass removed by erosion
Remnant Santa Clara Formation
Present valley floor
Penvault (SCVWD)
Clearwell (SCVWD)
Terminal Reservoir (DWR)
Elevation in feet
1800
1600
1400
1200
1000
800
600
400
200
0
0
800
1600
2400
3200
4000
4800
5600
6400
7200
8000
8800
9600

Note: 1) All schematics not to scale, except No. 6
2) File: W:\geology\nelson\coreldraw\cdr\penitencia\pencros4.cdr

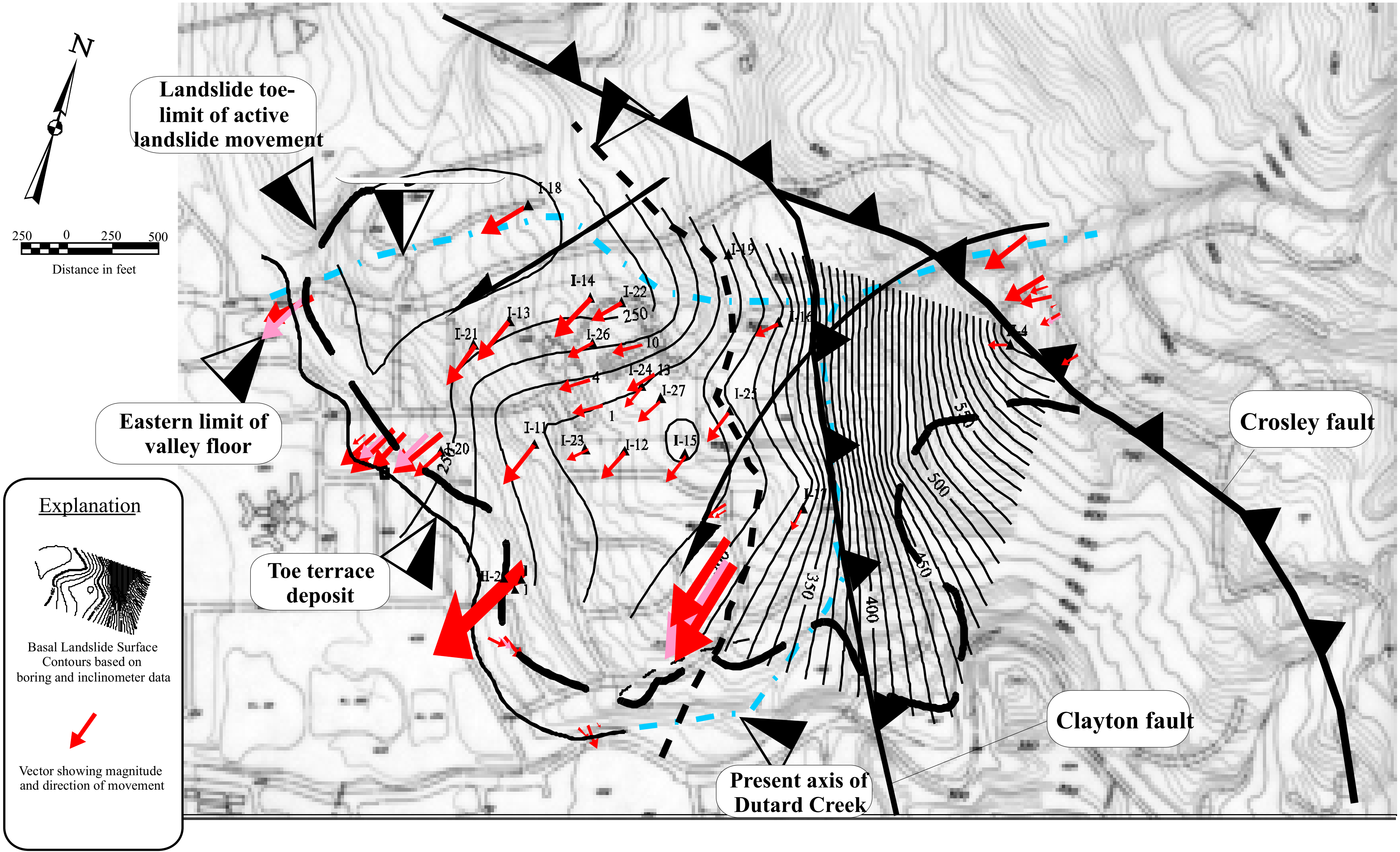
DATE	11/29/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Conceptual Landslide Evolution
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
41


N
250
0
250
500
Distance in feet
Landslide toe-limit of active landslide movement
Eastern limit of valley floor
Toe terrace deposit
Present axis of Dutard Creek
Clayton fault
Crosley fault
I-18
I-19
I-14
I-22
I-13
I-21
I-26
I-16
I-4
I-24
I-27
I-25
I-11
I-23
I-12
I-15
I-20
I-17
H-2
250
10
13
4
1
550
500
450
400
350
Explanation
Basal Landslide Surface Contours based on boring and inclinometer data
Vector showing magnitude and direction of movement

File:W:/geology/penitencia creek landslide/figures/fig42.srf

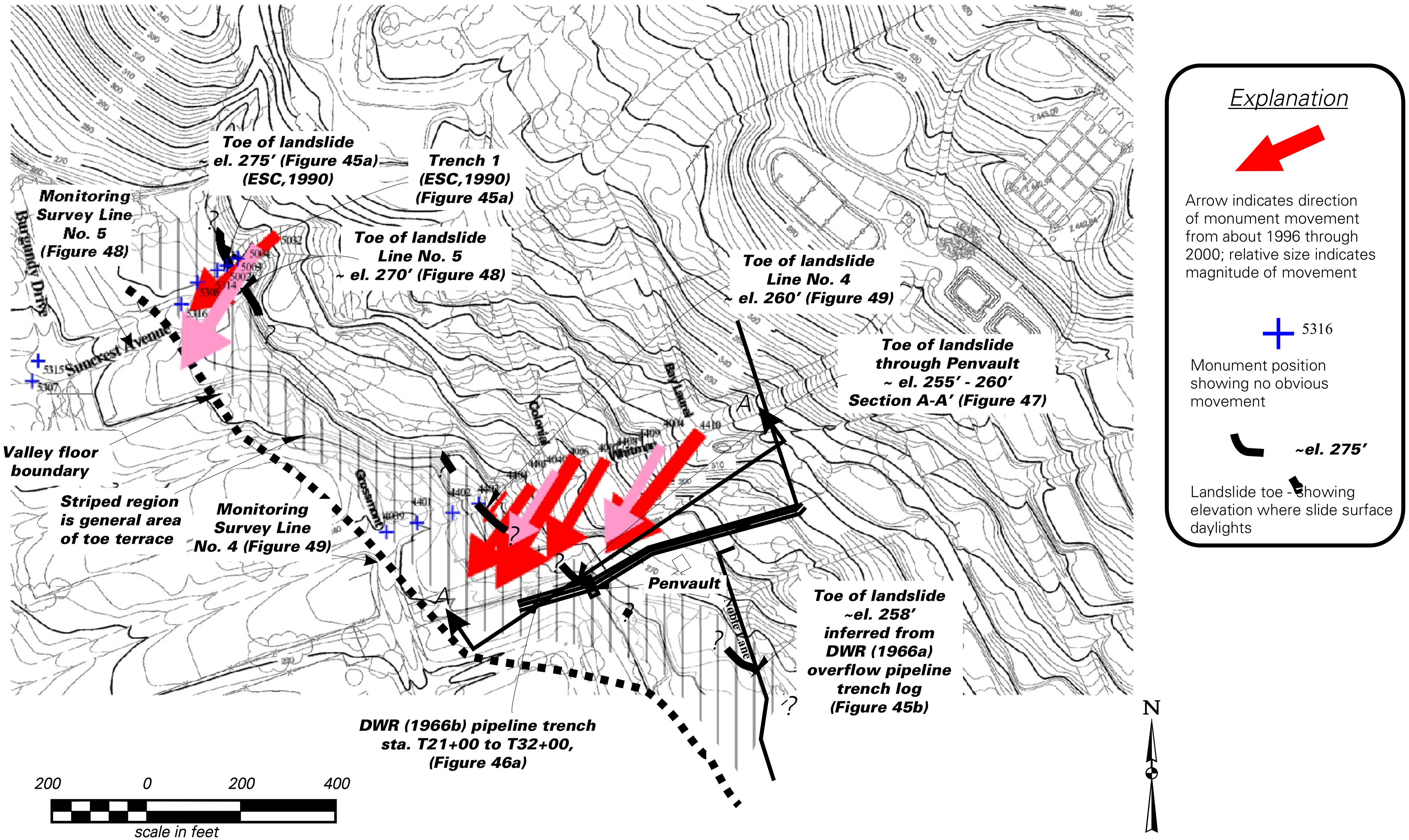
DATE	10/5/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Interpretive Contour Map of
Basal Landslide Shear Surface
Based on Subsurface Data
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"= 500'

FIGURE
42


Explanation
Arrow indicates direction of monument movement from about 1996 through 2000; relative size indicates magnitude of movement
5316
Monument position showing no obvious movement
~el. 275'
Landslide toe - showing elevation where slide surface daylights
Toe of landslide el. 275' (Figure 45a) (ESC,1990)
Trench 1 (ESC,1990) (Figure 45a)
Monitoring Survey Line No. 5 (Figure 48)
Toe of landslide Line No. 5 ~ el. 270' (Figure 48)
Toe of landslide Line No. 4 ~ el. 260' (Figure 49)
Toe of landslide through Penvault ~ el. 255' - 260' Section A-A' (Figure 47)
Valley floor boundary
Striped region is general area of toe terrace
Monitoring Survey Line No. 4 (Figure 49)
Penvault
Toe of landslide ~el. 258' inferred from DWR (1966a) overflow pipeline trench log (Figure 45b)
DWR (1966b) pipeline trench sta. T21+00 to T32+00, (Figure 46a)
Burgundy Drive
Suncrest Avenue
Grossmont
Colonial
Bay Laurel
Whitman
Noble Lane
A
A'
?
5315
5307
5308
5314
5316
5004
5003
5002
5032
4401
4402
4400
4404
4409
4408
4406
4410
200
0
200
400
scale in feet
N

Notes: 1) Elevations based on NAVD 1988
2) File: W:\Geology\Penitencia Creek Landslide\figures\Fig43.cdr

DATE	4/25/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

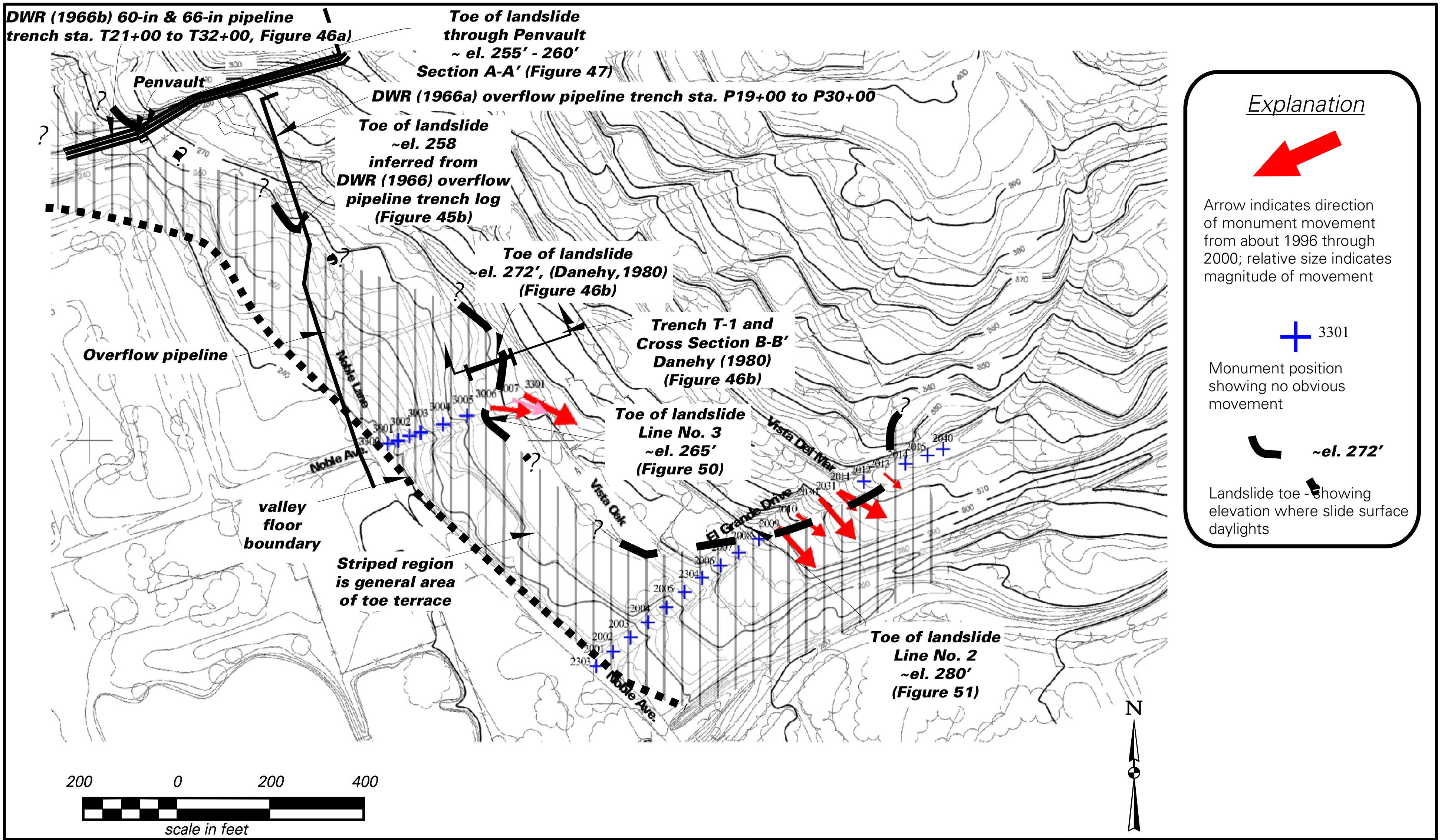
Santa Clara Valley Water District

Toe of Landslide Analysis
Northern Study Area

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=200'

FIGURE
43


DWR (1966b) 60-in & 66-in pipeline trench sta. T21+00 to T32+00, Figure 46a)
Toe of landslide through Penvault ~ el. 255' - 260' Section A-A' (Figure 47)
Penvault
DWR (1966a) overflow pipeline trench sta. P19+00 to P30+00
Toe of landslide ~el. 258 inferred from DWR (1966) overflow pipeline trench log (Figure 45b)
Toe of landslide ~el. 272', (Danehy,1980) (Figure 46b)
Trench T-1 and Cross Section B-B' Danehy (1980) (Figure 46b)
Overflow pipeline
Toe of landslide Line No. 3 ~el. 265' (Figure 50)
Noble Lane
Noble Ave.
Vista Oak
Vista Del Mar
El Grande Drive
valley floor boundary
Striped region is general area of toe terrace
Toe of landslide Line No. 2 ~el. 280' (Figure 51)
?
3301
3002
3003
3004
3005
3006
3007
2303
2001
2002
2003
2004
2005
2304
2006
2007
2008
2009
2010
2031
2011
2012
2013
2014
2015
2040
N
200
0
200
400
scale in feet
Explanation
Arrow indicates direction of monument movement from about 1996 through 2000; relative size indicates magnitude of movement
3301
Monument position showing no obvious movement
~el. 272'
Landslide toe - showing elevation where slide surface daylights

Notes: 1) Elevations based on NAVD 1988
2) File: W:\Geology\Penitencia Creek Landslide\Figures\Fig44.cdr

DATE	4/25/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

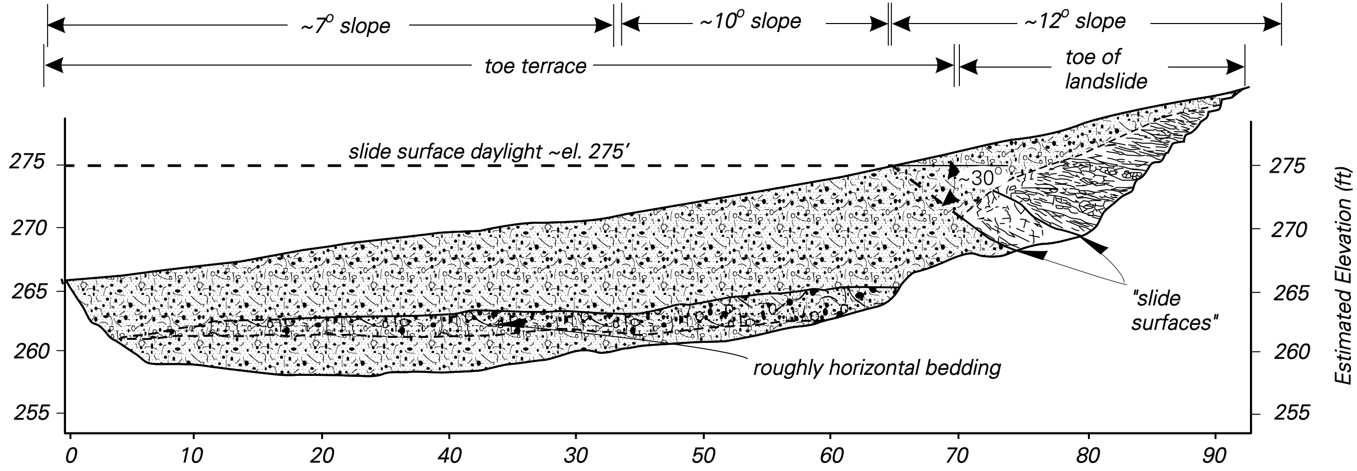
Santa Clara Valley Water District

Toe of Landslide Analysis
Southern Study Area

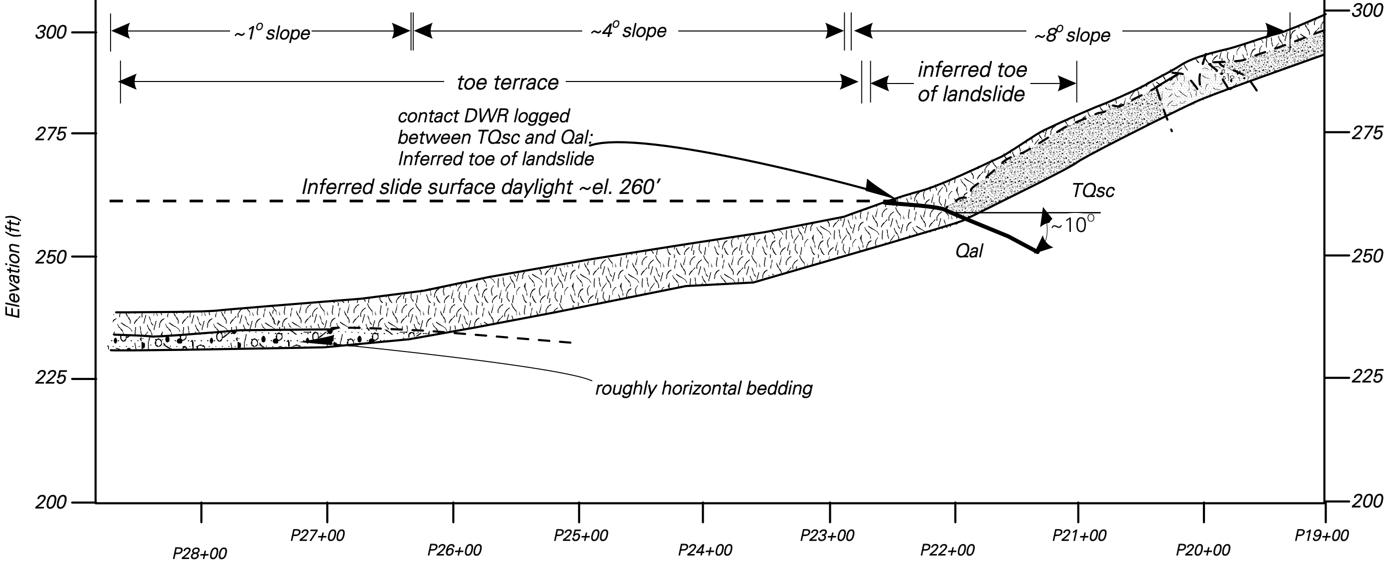
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=200'

FIGURE
44



45a) Earth Systems Consultants Trench 1 (1990) - Suncrest Ave.



~1° slope
~4° slope
~8° slope
toe terrace
inferred toe of landslide
contact DWR logged between TQsc and Qal; Inferred toe of landslide
Inferred slide surface daylight ~el. 260'
TQsc
~10°
Qal
roughly horizontal bedding
300
275
250
225
200
Elevation (ft)
P28+00
P27+00
P26+00
P25+00
P24+00
P23+00
P22+00
P21+00
P20+00
P19+00
Station (ft)

45b) DWR (1966a) - Overflow Pipeline Trench Log

Explanation

Fill patterns

- ▨ fat clay
- ▨ lean clay & silty clay
- ▨ gravelly, clayey silt to gravelly, cobbly silty clay
- ▨ sandy clay & clayey sand
- ▨ clayey gravel

Symbols

Qc	Colluvial soils
Qal	alluvial soils
TQsc	Santa Clara Fm

Notes: 1) estimated angles added by J.L.Nelson
2) angles corrected for exaggerated scale
3) elevations on original logs assumed to be NGVD 1929, corrected to NAVD 1988 by adding 2.75 feet.
4) as on original of DWR log vertical scale 4x horizontal scale
5) view of logs is opposite of original log for comparison purposes
6) logs generalized by grouping soil types (i.e. all CLs in 1 group)
7) file: W:\geology\penitencia creek landslide\figures\Fig45.cdr

DATE 5/1/01
DESIGN JLN
DRAWN JLN
CHECKED RLV

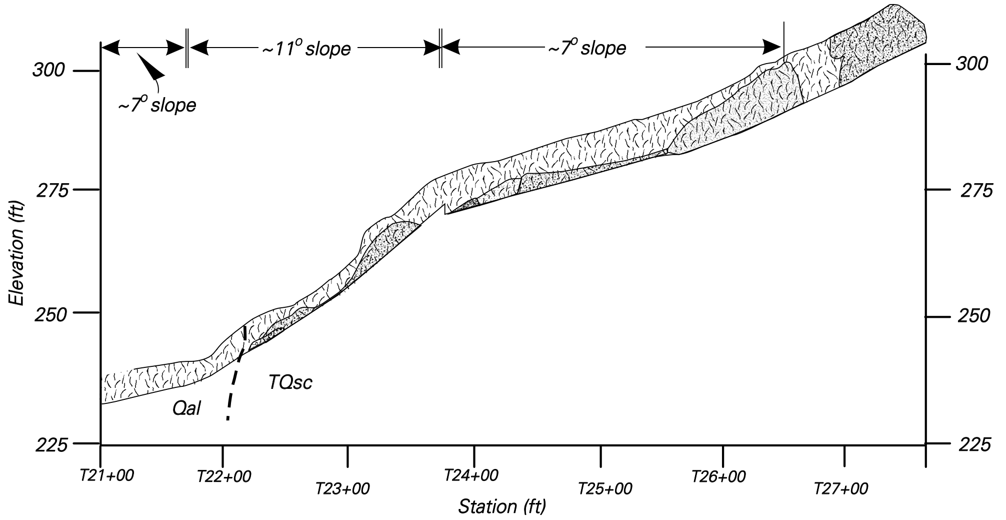
Santa Clara Valley Water District

Trench Logs Across Landslide Toe
Earth Systems Consultants, 1990
& DWR, 1966

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE as shown

FIGURE 45



46a) DWR (1966b) - 60-in & 66-in Pipeline Trench Log

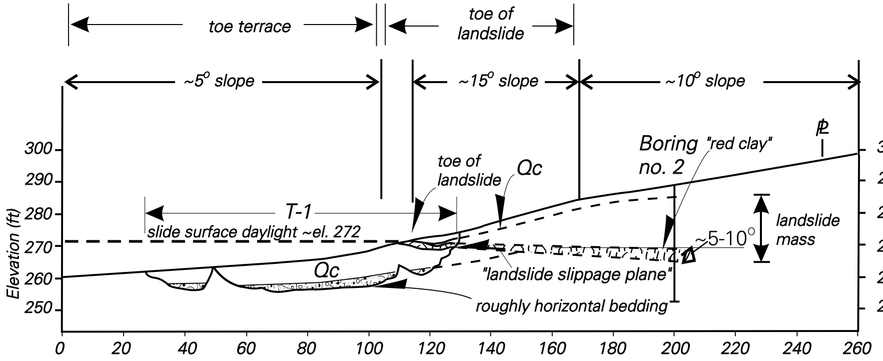
Explanation

Fill patterns

- ▨ fat clay
- ▨ lean clay & silty clay
- ▨ gravelly, clayey silt to gravelly, cobbly silty clay
- ▨ sandy clay & clayey sand
- ▨ clayey gravel

Symbols

Qc	Colluvial soils
Qal	alluvial soils
TQsc	Santa Clara Fm



46b) Edward A. Danehy (1980) - Trench Log of T-1 and Cross Section B-B' - Noble Lane and Noble Ave.

Notes: 1) estimated angles added by J.L.Nelson
2) angles corrected for exaggerated scale
3) elevations on original logs assumed to be NGVD 1929, corrected to NAVD 1988 by adding 2.75 feet.
4) as on original of DWR log vertical scale 4x horizontal scale
5) view of DWR trench log is opposite of original log for comparison purposes
6) logs generalized by grouping soil types (i.e. all CLs in 1 group)
7) file: W:\geology\penitencia creek landslide\Figures\Fig46.cdr.

DATE 5/1/01
DESIGN JLN
DRAWN JLN
CHECKED RLV

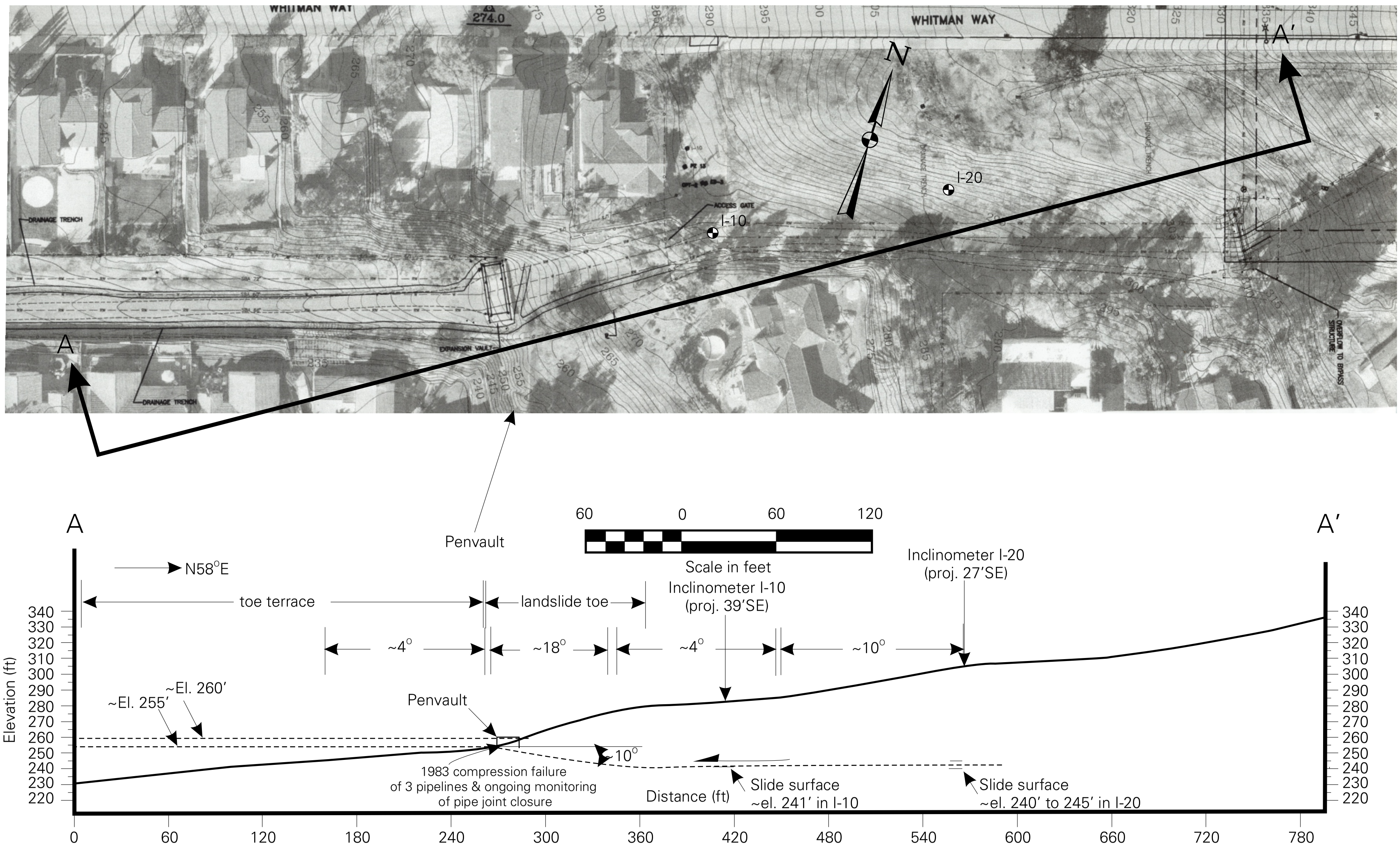
Santa Clara Valley Water District

Trench Logs Across Landslide Toe
DWR, 1966 & Danehy, 1980

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE as shown

FIGURE 46



Notes: 1) Elevations based on NAVD 1988
2) File: W:\Geology\Penitencia Creek Landslide\Figures\Fig47.cdr

DATE *5/16/01*
DESIGN *JLN*
DRAWN *JLN*
CHECKED *RLV*

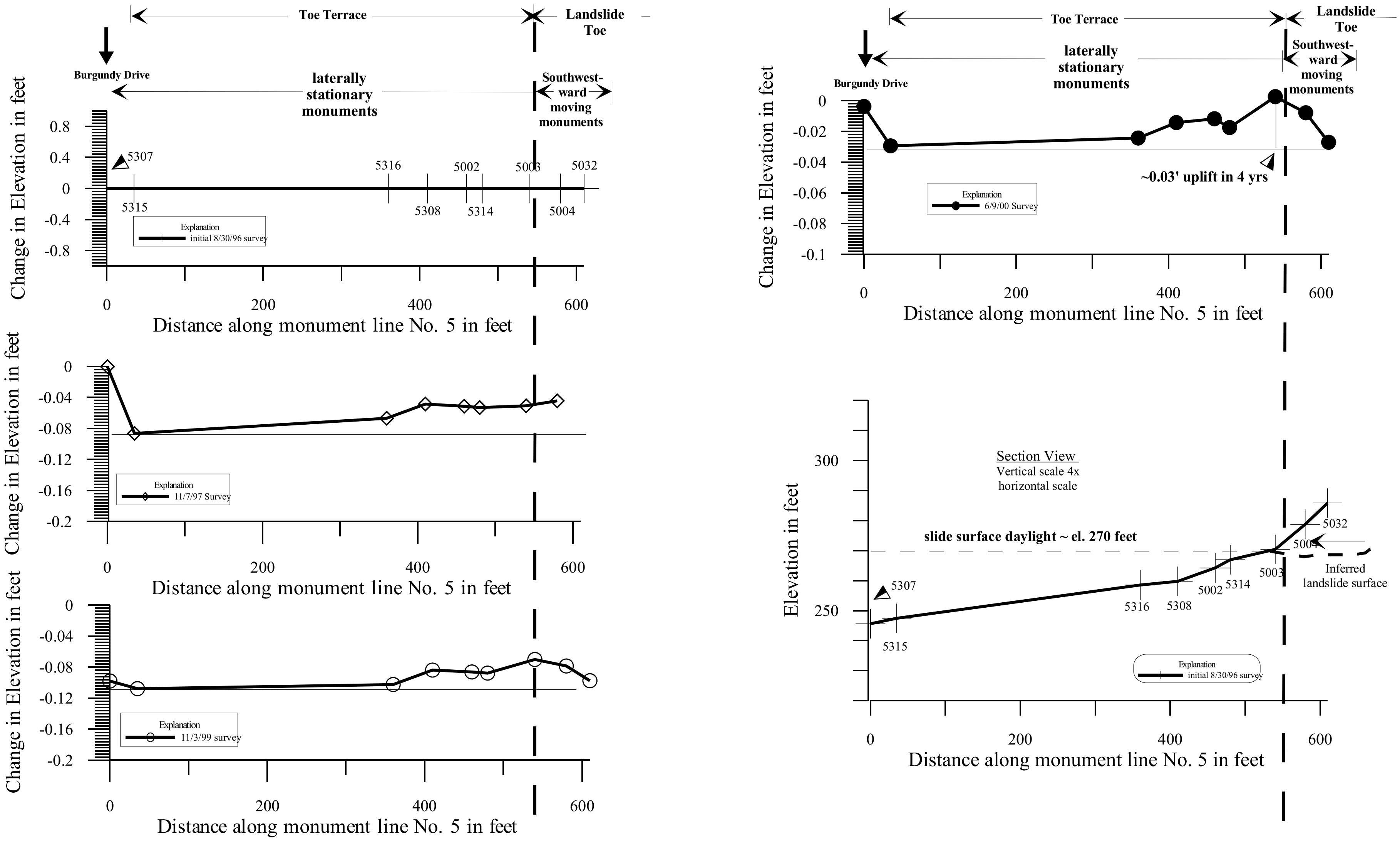
Santa Clara Valley Water District

Toe of Landslide Analysis
Penvault Area

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE *1″=60′*

FIGURE *47*



Notes:
1) changes in elevation exaggerated 1000x horizontal scale
2) File: W:\geology\Penitencia Creek Landslide\Figures\Fig48.grf

DATE	4/22/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

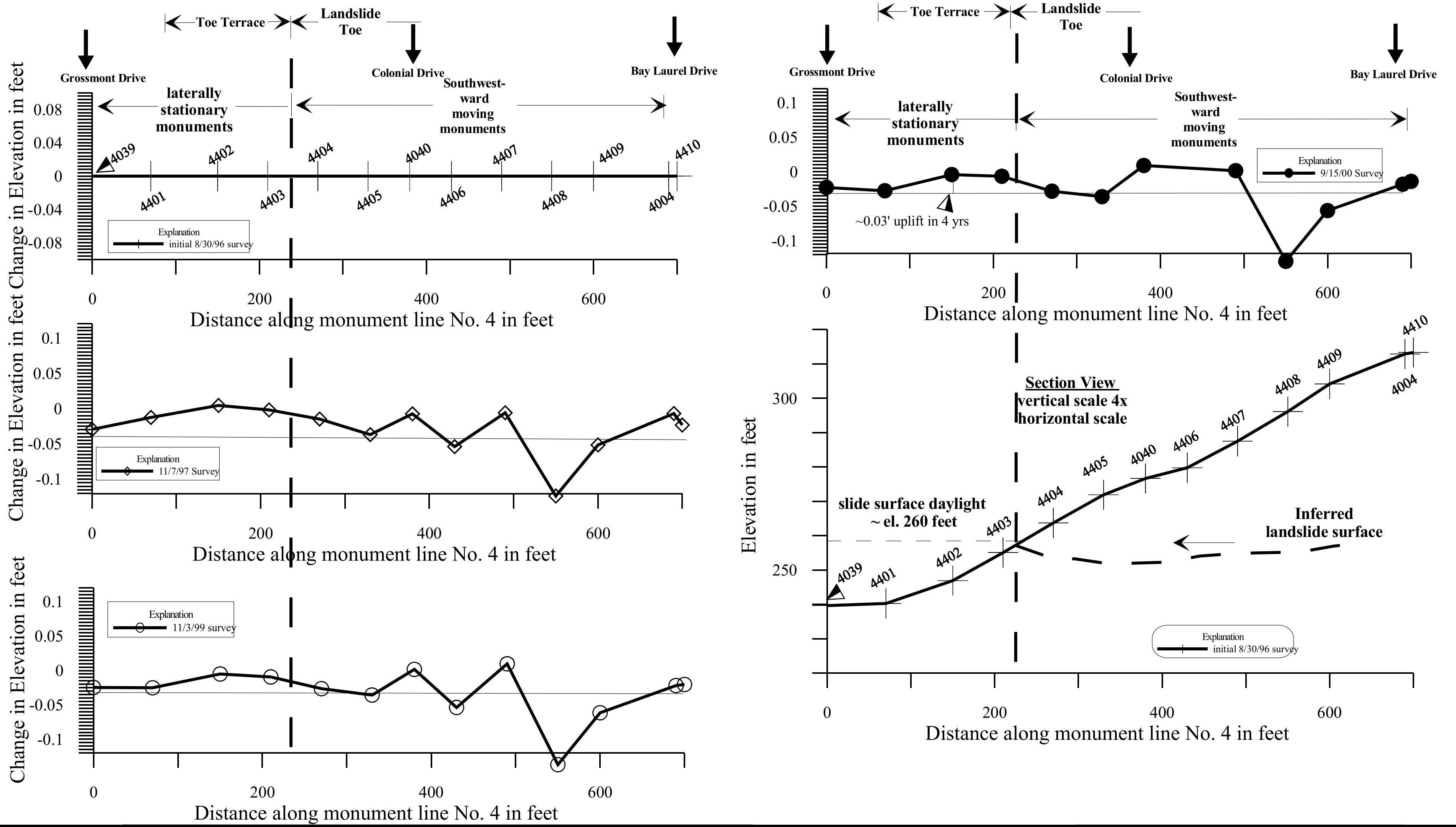
Santa Clara Valley Water District

Relative Elevation Changes and Section View
Monitoring Line No. 5, 8/96 through 9/00

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
48



Notes:
1) changes in elevation exaggerated 1000x horizontal scale
2) File: W:\geology\Penitencia Creek Landslide\Figures\Fig49a.grf

DATE	4/22/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

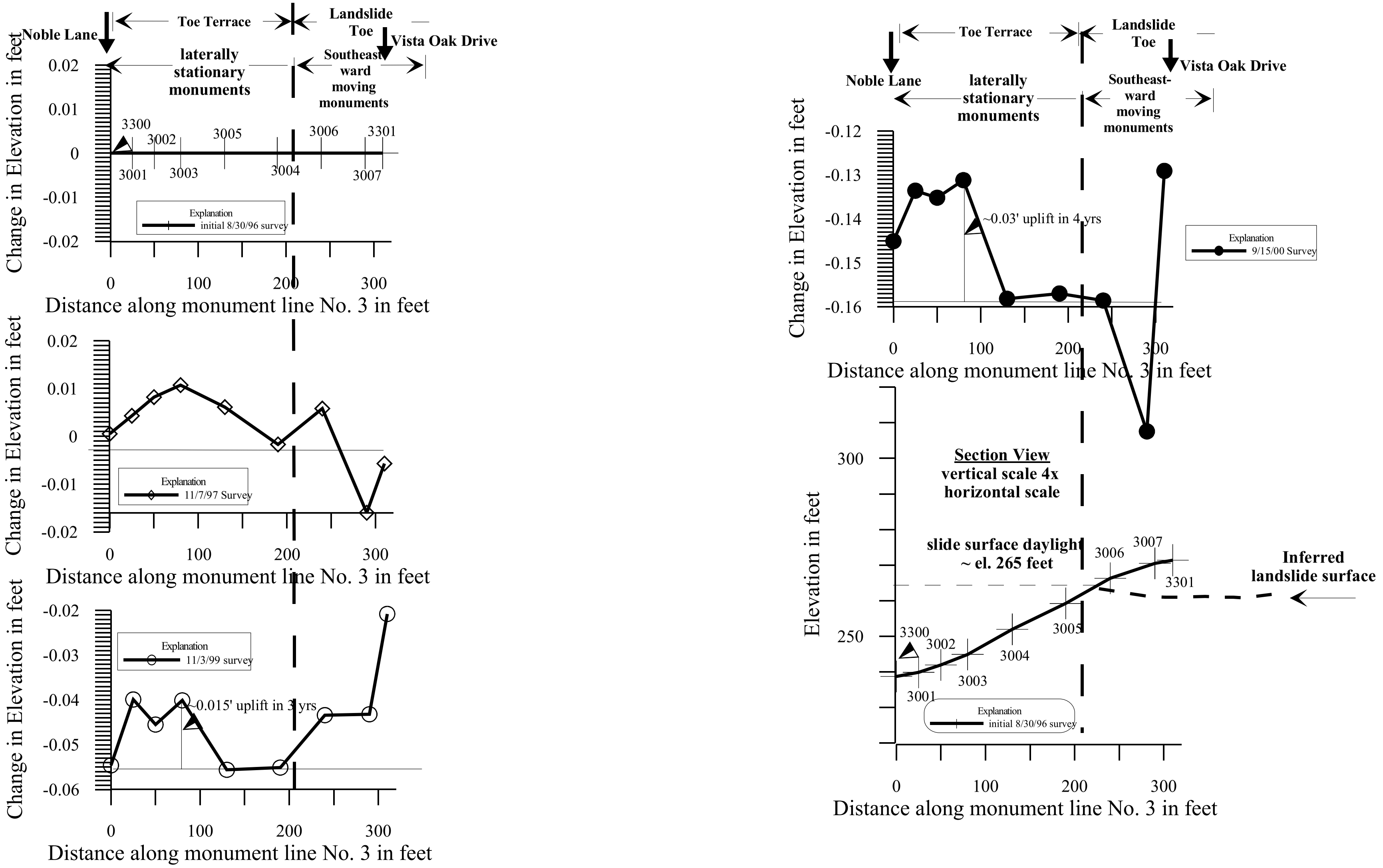
Santa Clara Valley Water District

Relative Elevation Changes and Section View
Monitoring Line No. 4, 8/96 through 9/00

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
49



Notes:
1) changes in elevation exaggerated 1000x horizontal scale
2) File: W:\geology\Penitencia Creek Landslide\Figures\Fig50a.grf

DATE	4/22/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

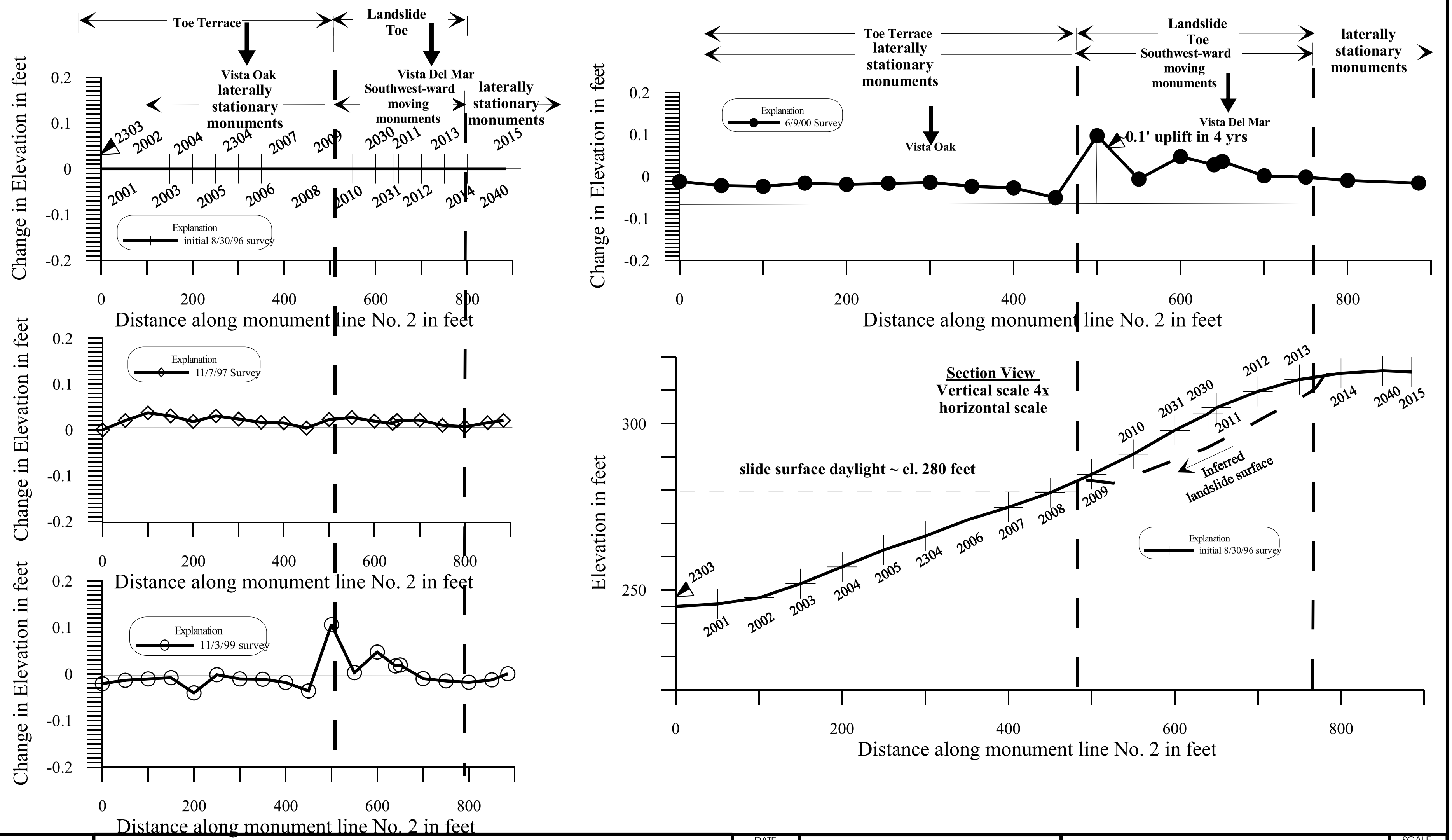
Santa Clara Valley Water District

Relative Elevation Changes and Section View
Monitoring Line No. 3, 8/96 through 9/00

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
50



Notes:
1) changes in elevation exaggerated 1000x horizontal scale
2) File: W:\geology\Penitencia Creek Landslide\Figures\Fig51a.grf

DATE	4/22/01
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

Relative Elevation Changes and Section View
Monitoring Line No. 2, 8/96 through 9/00

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
as shown

FIGURE
51

8.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our review of 28+ years of surveillance data, along with pertinent geologic and geotechnical aspects of the Penitencia Creek Landslide, we have developed the following conclusions and recommendations. The following sections summarize the principal findings of this investigation and present our current understanding of the mechanism, magnitude and rate of landslide movement. Since this report has been published following the completion of final design for the Phase 1 TWIP work, but prior to completion of the Phase 2 TWIP work, we recommend that the conclusions presented herein be thoroughly reviewed by the TWIP design team. Recommendations are presented that address future monitoring efforts and specific future technical analyses that appear warranted. The conclusions and recommendations are sequentially numbered and, where appropriate, have been grouped into broad general headings.

8.1 CONCLUSIONS

8.1.1 Description of Landslide and Mechanism of Failure

1. The Penitencia Creek landslide is a creeping landslide that probably developed about 18,000 to 20,000 years ago and is still active. The existing landslide occupies about 240 acres of land that includes the District's PWTP, DWR's Terminal Reservoir, and San Jose Water Company's Dutard Reservoir. The base of the landslide is relatively deep beneath the plant with the basal failure surface extending to depths of up to 270± feet below ground surface. Beneath the PWTP, due to the orientation and shape of the failure surface, the landslide mass is moving approximately horizontally. Because the plant is essentially moving as a unit with the landslide, very little differential movement is occurring. This explains why very limited structural damage has occurred at the plant since its original construction in 1973/74.

2. The landslide has developed and propagated within highly plastic, over-consolidated claystone units of the Santa Clara Formation. Results from x-ray diffraction and electron microscopy of the material obtained from the landslide surface indicate that the clay fraction (material less than 0.002 mm in size) is composed predominantly of smectite clay minerals. Due to the brittle nature of the claystones, a characteristic imposed by the heavy overconsolidation following original formation of these materials, they rapidly lose strength as they weather. This form of rapid strain softening, when combined with the presence of smectite clay minerals, results in the development of an unusually low residual friction angle along landslide failure surfaces. The average residual friction angle of the Santa Clara formation claystone material is between 5-8 degrees based on its plasticity, results from previous research, and actual shear strength testing of similar materials elsewhere. Because the residual strength friction value is very low and the slope of the ground surface of the landslide mass is close to the residual friction angle, it is highly likely that there is only one principal shear surface and that the majority of all landslide movement has occurred on this single surface.

3. Due to extensive erosion over geologic time, the current physical limits of the landslide represent a relatively small portion of the landslide's original size. During the past 18,000 to 20,000 years, much of the original headscarp, which appears to have originally extended upslope (easterly) almost one mile to the Hayward fault, has since been highly modified by erosion. Due to prolonged downhill displacement, the landslide mass has moved out onto the valley floor about 1,500 to 2,000 feet. It is quite likely that the landslide will continue to encroach on the valley floor for hundreds to possibly thousands of years, albeit at an ever decreasing rate of movement. Based on our hypothesis of the landslide mechanism, we believe that this movement will continue to occur over geologic time until the overall slope of the landslide mass becomes stable and achieves an overall average slope of between 5-8 degrees. Although it is not possible to estimate how much longer the landslide will remain active, we feel confident that it lacks the potential to propagate any further upslope.

8.1.2 Description of Landslide Damage

4. The major damage experienced at the District's PWTP since its construction in 1973/74 has been the failure in 1983 of the two large diameter pipes leading to the plant, and the large diameter pipe carrying the South Bay Aqueduct water to the DWR Terminal Reservoir. The compression failure damage to these pipes appears to have occurred at those joints closest to the landslide toe where the pipes pass from stable ground onto the moving landslide. Similar movement will continue to occur at slow but persistent rates in the future. To accommodate ongoing and future movement of the pipes, and thus prevent future pipe failure, the District has installed a specially designed pipe bellows to accommodate landslide movement. The limit of slip potential (3-3.5 inches) of the bellows has been exhausted and needs to be reinstated.

5. The 3-acre landslide behind (upslope) the DWR Terminal Reservoir continues to move. This landslide appears to have developed as a result of unsupported stress releases that developed shortly after DWR began excavation for the Terminal Reservoir in 1964. The resulting slide has remained active since that time. DWR continues to remove an estimated 100 cubic yards of slide debris per year to keep the debris from accumulating and adversely loading the steel tank of the reservoir. The possibility that the smaller 3-acre landslide could expand in size and propagate downslope beneath the PWTP site due to proposed excavation as a result of TWIP project appears to be extremely remote as long as proposed excavations are properly supported during construction.

8.1.3 Magnitude, Rate, and Orientation of Principal Landslide Movement

The following discussion relating the magnitude, rate, and orientation of movement pertains to the average section of the Penitencia Creek Landslide near the District's treatment plant facility. The much smaller landslide behind the DWR Terminal Reservoir is moving substantially faster than the larger slide. Also, portions of the larger slide closer to Penitencia Creek are moving at faster rates and at a different orientation than the main slide.

6. Based on an evaluation of the shear deflection of 28 inclinometers installed at various locations throughout the landslide, we have determined average rates of movement along the active landslide surface of between 0.1 and 0.4 in/yr. These rates are not constant either spatially or temporally over the lateral extent of the landslide. There appears to be a direct correlation between landslide movement and periods of heavier rainfall. Following periods of heavy rainfall, landslide movements of up to 1.0 inch have occurred over a period of several months. Increased rainfall may accelerate landslide movement for short periods of time (perhaps up to 6 months) due to a combination of adding weight to the mass and by increasing pore pressures, thereby decreasing the mobilized shear strength, along the landslide shear surface. Rapid response to significant rainfall strongly indicates that rainfall readily infiltrates the landslide mass, at least in certain locations along and adjacent to the upper sections of Dutard Creek and Sierra Creek. Inclinometer data also strongly indicates that the landslide may become temporarily dormant during summer months or periods of lower rainfall. The rapid response of landslide movement to rainfall may, in part, be due to the presence of Dutard Creek and Sierra Creek, two creeks that run from the landslide headscarp area to the valley floor.

7. Based on DWR survey readings of selected monitoring points on the landslide surface using electronic distance measuring equipment (EDM), computed overall average landslide movement rates near the plant facility of between 0.3 and 0.5 in/yr. are computed for the period from 1972 to 1997. During the period from 1984 through early 1986, during which period the Morgan Hill earthquake and an unusually heavy rainfall (about 10-inches in 3 months) occurred, the overall rate of landslide movement jumped to between 0.6 and 2.0 in/yr. for relatively short periods but have since returned to their pre-earthquake rates.

8. Based on GPS survey of brass monuments at the four corners of the plant, which began in 1991, the surface movement rates at PWTP have averaged about 0.28 in/yr.

9. For the six street monument arrays located at various orientations over the landslide, movement rates are much less consistent than other types of measurement, ranging from 0.05 to 0.58 in/yr. There are concerns regarding the stability of the GPS monument points. Measures that address these concerns are covered under recommendations.

10. Analysis of the three methods for determining landslide movements namely inclinometer, EDM, and GPS, have provided computed directions of movement adjacent to the PWTP site that varies between due west to southwest. These directions are entirely consistent with existing topography. Other parts of the landslide, especially on the southeast side, are moving differently. While developing an interpretive contour map of the basal landslide shear surface based on elevations at which failure is occurring in inclinometers, we discovered that the actual landslide surface is dipping almost due north beneath the PWTP because the landslide is following topography of the ancestral Santa Clara Valley. The lines of equal elevation defined by the topography of the basal shear surface directly beneath the PWTP are roughly perpendicular to the ground surface contours and parallel to the direction of landslide movement. Our analysis indicates that

contours of the basal shear directly beneath the PWTP run roughly parallel the long axis of the plant. This favorable configuration of the landslide surface relative to the ground surface is one of the primary reasons why there is such limited ground distortion at the site.

8.1.4 Magnitude and Rate of Secondary Landslide Movement

11. Our interpretation of inclinometer data suggests that secondary movement of the landslide (e.g. at points other than the principal landslide surface) is occurring at rates at least an order of magnitude less than the rate of movement along the main slide shear surface. It is not clear whether the secondary movements are a result of landslide adjustment along previous or historic landslide planes or the result of movements of highly expansive soils contained within the slope material in the form of slope creep. Secondary downslope movement rates of between 0.01 and 0.06 in/yr. have been monitored in the inclinometer borings at numerous locations and elevations. Since these movements are apparently randomly oriented, we conclude that there does not appear to be any direct correlation of the secondary movements to suggest the possible development of another preferential slide plane.

Ground Water Levels Beneath and within the Landslide

12. The shallowest piezometric surface at the site occurs just below the west side of the Terminal Reservoir and is located about 15 feet below ground surface. It is possible that this higher ground water level may have been influenced as a result of leakage from the Terminal Reservoir that has subsequently been repaired. The moderately high ground water level within portions of Penitencia Creek Landslide is most likely due to the probability that either or both of Dutard Creek and Sierra Creek acts as the principal source for recharging the ground water within the landslide.

13 In assessing future work at the site, it appears highly likely that groundwater will be encountered during the excavation of the Ozone Generator building during the Phase 2 TWIP work. The maximum required depth of excavation for this structure will be about 31 feet and analyses suggest that there may be up to 16 feet of water pressure at the base of the excavation. Additional piezometers are warranted to further evaluate this issue.

14. Based on our analysis of subsurface information, it appears that a large-prism of older alluvial valley fill lies beneath the southwestern portion of the landslide. We estimate that during the life of the landslide it has encroached out onto the original valley floor about 1,500 to 2,000 feet. In addition, stream sediments deposited in an ancestral channel of Dutard Creek pass beneath the northern portion of the landslide. These sediments make up what appears to be a fairly large basin that collects a significant amount groundwater. This groundwater represents the deepest piezometric surface encountered in piezometers and inclinometer borings that penetrate through the landslide mass. The lower piezometric surface, at least directly under the plant, is between 120-150 feet lower than the piezometric surface present within the landslide mass. Based on the response of some of the piezometers we speculate that where the piezometer and

Inclinometer drill holes have penetrated the basal landslide surface there may be some beneficial release of the higher piezometric pressures provided by drainage through the borings into the lower piezometric surface. If such a process is occurring, it would be very difficult to determine how significant or widespread this postulated drainage phenomenon may be.

8.1.6 Other Geologic Hazards

15. The Penitencia Creek landslide is in close proximity to several active faults of the San Andreas fault system. The Clayton, Crosely, and Berryessa faults consist of a series of imbricate thrust faults located less than 1/2 mile east of the site. The Hayward fault is located about one mile uphill (east) from the site. Other active faults in the region, including the San Andreas and Calaveras faults are sufficiently close to the site to present potential hazards. Using current earthquake motion attenuation relationships by Abrahamson and Silva (1997), we calculated the maximum Peak Ground Acceleration (PGA) potential at the site to be about 0.69 g if a magnitude 6.4 earthquake were to occur on the nearby Southeast Extension of the Hayward fault.

16. Results from x-ray diffraction and electron microscopy indicate that the clay fraction (less than 2 mm) of soils from the landslide surface is composed predominantly of smectite clay minerals. Other geotechnical data indicates that these are predominantly sodium smectite clay minerals, which are one of the most active clay minerals. Based on estimates from the plasticity index of clay samples obtained from the slide mass, and results inferred from direct shear test data on an intact shear surface from Saratoga, California, we estimate the residual friction angle along the basal shear surface of the Penitencia Creek Landslide to be as low as 6 to 8 degrees.

17. Driving forces controlling active creep of the Penitencia Landslide need not be great because the landslide shear surface appears to be nearly horizontal and possesses low residual friction angles. As long as driving forces do not increase significantly, the authors believe the landslide will continue to creep at relatively low rates. The major forces that could increase driving forces appear to be earthquakes, and changes caused by increased groundwater levels in the landslide mass. The impact of several moderate earthquakes since the plant was constructed has caused only relatively minor earthquake induced displacement events; however, it is noted that the peak acceleration values of recent earthquakes are substantially less than the peak ground acceleration value expected by a potentially larger earthquake on the nearby Hayward fault. Previous dynamic analyses by District consultants (Woodward-Clyde Consultant, 1993) were performed to simulate potentially larger earthquakes of M=7 on the Hayward fault. Results of these analyses suggest that permanent earthquake-induced displacements of the PWTP site could amount to between 1 foot and 6 feet depending on antecedent conditions of the landslide materials. Based on our assessment described in this report, it is possible that the unique and favorable orientation of the major slide plane under the plant could moderate these expected downslope distortions. We recommend that the previous dynamic analyses be reviewed in light of the findings presented in this report.

8.2 RECOMMENDATIONS

1. Readings of the active piezometers, inclinometers, and surface monuments at the plant and surrounding areas should be continued quarterly, except as noted below.

2. Based on the favorable results presented in this report, it appears that future surveillance efforts should focus on the use of GPS survey techniques in conjunction with installation of at least one in-place automated inclinometer (summarized in Item 5, below). We encourage DWR to continue their EDM surveys.

3. Since installation of new survey monuments on the treatment plant will be required following the current Phase 1 TWIP improvements, an assessment and cost effective analysis should be performed to assess the merits of installing a permanent GPS survey system at the site. Such a system would preclude the need for quarterly surveys. The equipment type and specifications for an appropriate GPS system should be coordinated with the District's Land Surveying and Mapping Unit.

4. New, more stable, surface monuments should be installed along the existing street survey lines. Selected existing "shiners" that were installed associated with the street surveys should be replaced with permanent monuments consistent with GPS monitoring requirements. Consequently, not every "shiner" needs to be converted to a GPS quality monument. The monument type, location, and installation requirements should be coordinated with the District's Land Surveying and Mapping Unit.

5. The historic collection of inclinometer data using a manual probe during the last 28 years has given us a fairly complete understanding of the overall landslide surface geometry and general rates of landslide movement. A significant limitation to performing quarterly manual surveys, however, is that we are not able to fully understand the relationship between landslide movement and transient events such as intense rainfall and earthquakes. Consequently, we recommend that at least one new inclinometer boring be installed with an in-place automated inclinometer system. Specifically, a new inclinometer boring should be installed adjacent to an older boring where the depth to the main landslide shear surface is known. A permanent inclinometer (tiltmeter) should be placed at the landslide shear surface to continually (daily) measure the deformation of the inclinometer casing. The in-place inclinometer would be connected to an automated datalogger and modem system similar to the existing one that monitors pipe-coupling closure within the Penvault.

5. We recommend that collection of quarterly manual inclinometer data along the main landslide surface be discontinued once the currently active inclinometer borings are no longer functioning. However, the upper sections of selected inclinometer borings should continue to be read to monitor suspected secondary slide movement.

6. The feasibility of installing alarm capabilities for future in-place inclinometers (Item 5) and/or existing TDR cables should be assessed. The alarm system would alert District personnel of landslide movement that exceeded a specified critical amount.

8. For general monitoring of existing Time Domain Reflectometry TDR cables placed within DWR inclinometer borings (DWR-6, DWR-7, and DWR-8) and one District inclinometer (I-20) the District should purchase a (TDR) readout device. This will eliminate the need for continued extensions of sub-contract work to read the cables and the device could be used in a future alarm system. The approximate cost for a TDR readout box is about $5,000.

9. A strong motion accelerometer should be installed (at a location yet to be selected) in the Control Building to record future earthquake motions. A question currently exists regarding whether earthquake motions will be attenuated as they propagate through the underlying geologic formations. The installation of a strong motion accelerometer at the plant will be very helpful in resolving earthquake ground response issues and provide valuable information regarding estimates of possible plant movement during future earthquakes. At Lenihan Dam, a strong motion accelerometer (Etna model by Kinemetrics), including the cabling, battery, and solar panel, was installed for about $7,500. These units are completely self contained and should not require any maintenance from plant staff.

10. The existing collapsible bellows for the two District and DWR pipe located in the Penvault near the toe of landslide are near the end of their 3-3.5-inch maximum travel. The bellows should be re-positioned to their maximum opening and the existing closure measurement devices should be reinstated. Consideration should be given to monitoring the rate of bellows closure for the SBA. The possibility to provide alarm capabilities for the movement devices should also be assessed.

11. Based on the geologic and geotechnical findings presented in this report, new static and dynamic slope stability analyses of the Penitencia Creek Landslide should be performed. To our knowledge, there has never been a static stability analysis performed on the main landslide. Such an analysis could be very instructive on the sensitivity of pore pressure in activating landslide movement. Dynamic stability analyses should be performed to improve our understanding of the potential for earthquake-induced landslide movement for several assumed groundwater levels representative of historical ranges in pore pressure and several different ground motions representative of design earthquakes on the Hayward, Calaveras and San Andreas earthquake faults. Such an analysis would produce a realistic range of possible landslide movements during earthquakes based on antecedent conditions. The new dynamic analyses should be performed using newer nonlinear analytical methods.

12. Based on the relatively high ground water acting downhill from the Terminal Reservoir, the District's consultant, as a part of the Stage 2 TWIP work, recommended the installation of one or more piezometers to monitor pore pressure associated with the required construction. Based on the ground water conditions discussion presented in this report, it is recommended that several vibrating gauge piezometers be installed prior to commencing with the Stage 2 TWIP work. A minimum of two piezometers, one on each side of the Ozone Generator building, should be installed to a depth of at least elev. 380

feet. These piezometers can be installed either by driving them using a CPT rig, or by conventional drill rig, whichever is most appropriate considering the existing construction conditions. The installation of these piezometers will have to be coordinated with the ongoing Stage 1 TWIP work.

13. Future surveillance reports should be prepared periodically as needed and summarize the performance of the various instruments and measuring points that constitute the surveillance instrumentation for the Penitencia Creek Landslide.

9.0 REFERENCES

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APPENDIX A

ANNOTATED REFERENCE LIST AND CHRONOLOGY

This appendix presents an annotated reference list and chronology of work performed for or by the Santa Clara Valley Water District pertaining to the geologic and geotechnical conditions at the District's Penitencia Water Treatment Plant. During the early stages of this report preparation, it was decided that an inventory of studies, investigations, and reports pertaining to the site would be a valuable tool that would serve two purposes: 1) in assembling and screening from the vast accumulation of related records in District files, and identifying relevant documents and literature available on site, it would facilitate the production of this report by providing pertinent historical perspective and availing certain literature essential to this report's content; and 2) it would provide future readers interested in the subject a single, comprehensive summary of information produced over the years pertaining to the plant's underlying geologic foundation conditions and the associated concerns for the plant's long-term stability.

The sources of information included in the listing are not presumed to represent a complete nor exhaustive collection. There are undoubtedly certain documents that could and probably should be included, but what is presently included can be considered reasonably comprehensive for the scope of the report.

Most of the original information referenced in this appendix is located in District files. The documents are listed chronologically beginning with the March 1962 investigation report by Kennedy Engineers and runs through the August 2000 geotechnical investigation and draft report by Harza Engineering. A number of references to significant events are included, although most entries are for reports. Each report entry includes a reference number in the **Date** column, and the referenced notes are defined on the first page. As related to those notes, further explanation of sources and selection of information is provided in the next paragraph.

A measure of subjectivity was unavoidable in selecting which information would be appropriate to include in the Appendix A list. As such, during the initial stages of reviewing the various reports, certain documents stood out as being benchmark investigations by virtue of their sheer relevance to the scope of this report. Several of those reports ultimately became useful compendiums from which other references and information could be checked or cross-checked. Where time did not permit a more thorough physical search for referenced reports deemed relevant for inclusion, their absence in District files did not preclude their inclusion, and the notes defined on the first page acknowledge the source of their relevance.

It is important to note that there has been considerable land surveying work done by the District's surveying unit, and also by the State Department of Water Resources and others, to determine the displacements of numerous survey monuments at various locations around the landslide. Although the body of land survey information has not received mention in the following reference list and chronology, that fact by no means diminishes the importance of that information. It has not been included because the work has not been conducted as separate, distinct projects, nor has any given survey culminated as a self-standing investigation and report.

Penitencia Creek Landslide
Annotated Reference List and Chronology

Notes in Date column:

1 Document is in District files or library.

2 Document not located in District files, but copy is included in *Environmental Impact Analysis of the Penitencia Water Treatment Plant and Evergreen Canal* , by Murray-McCormick, 1972.

3 Document not located in District files, but discussed or referenced with conclusions as reported in *Engineering and Geological Study, Penitencia Water Treatment Plant Expansion,* by Wahler and Associates 1982, for Metcalf & Eddy, Inc.

4 Document not located in District files, but discussed or referenced with conclusions as reported in *Penitencia Water Treatment Plant Site Study,* by Woodward-Clyde, 1993.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Mar 1962 1	Kennedy Engineers	*Water Treatment for South Bay Aqueduct Supply to Santa Clara County*	"The site lies in Santa Clara gravels and clays and would require moderate cuts to develop a level site. Soft overburden is deep and the Santa Clara formation is unconsolidated. Although the foundation materials are soft, it is believed that the site is satisfactory geologically."	None. ". . . some variation in design might be necessary after the site is explored by drilling..."
Nov 1963 2, 3, 4	Marliave, Burton For Kennedy Engineers	*Geologic conditions at Penitencia Creek site*: Consultant's letter report to Kennedy Engineers.	Recommended piezometer installation; "The site is believed to be geologically and topographically a suitable location for the proposed treatment plant."	Letter report only, one + page in length; no reference to exploratory borings.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Jan. 1964 2, 3, 4	Robert S. Cooper and Associates	*Foundation Investigation, Proposed Penitencia Water Treatment Plant*: Consultant's report to Kennedy Engineers, for the Santa Clara Valley Flood Control and Water Conservation District.	"Purpose . . .to evaluate soil and foundation conditions at the site and develop recommendations for cut and fill slopes, site grading operations, allowable bearing pressures and settlements of structures. The suitability of the site from the standpoint of engineering geology was evaluated by your engineering geologist, Mr. Burton H. Marliave."	17 borings, 20 to 100 ft depths; boring logs and lab tests of undisturbed samples; piezometers installed in borings 4, 8, 16, and 17; water levels were observed on 1/6/64 and 3/4/64 and noted on the boring logs (copy of report contained in March 1972 Environmental Impact Analysis by Murray-McCormick.
Apr 1964 1	Calif. Dept. of Water Resources	*Geologic Data, Terminal Reservoir, Terminal Facilities, Santa Clara County, Calif., Project Geology Report D-30*	Summaries of subsurface geologic exploration data and tabulations of primary laboratory test data available prior to March 1, 1964	Four auger holes along intake pipeline drilled Sept 1963, no soil tests (either not made or not presented in report); Five auger holes and three core holes at TR site: Lab tests for USCS classing only (no shear). Seven boring logs shown on plan, depths to113.4, all CH except one boring with GW at ~425' elev; water levels shown on 9-4-63 and on 12-17-63

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Jun 1966 1	Calif. Dept. of Water Resources	*Final Geologic Report on the construction of the South Bay Aqueduct, Terminal Facilities; Project Geology Report C-12, Appendix 5 to final construction report on South Bay Aqueduct terminal facilities*. (Terminal Reservoir construction was complete March 1966)	Terminal reservoir (TR) is at toe of active landslide; shallow GW; fissures up to 30' deep in slope above tank. Cut slope failure d/t excavation; 4 inclinometers installed by 1/65, aluminum casing, permeable backfill, 2 GW observation wells installed; also install. of horizontal drains, but ineffective due to high clay content of soil.	Preconstruction work incl. nine 28-inch bucket auger holes & three core holes; lab tests; info in previous interim reports of 1963, and 1966; add'l borings during constr incl 4 auger holes in 11/64, 4 rotary holes w/ slope indicator casings 12/64 and 1/65; geologic logs of pipeline trench excavated across toe of landslide. Geologic logs of pipeline trench are included in this report.
1966 3	Gribaldo, Jacob, Jones, and Assoc.	*Soil Investigation for Sunset Hills Subdivision, Unit No. 4, Tract No. 3083, San Jose, California*	Soils investigation report for proposed 22 lot development on property later purchased by SCVWD	Seven shallow borings (max. depth 18 ft)
Mar 1968 1	Gribaldo, Jacob, Jones, and Assoc.	*Report to San Jose Water Works, San Jose, California, on Seepage Investigations for Dutard Reservoir*	Tentative conclusions from incomplete investigation; purpose of investigation was to determine if underground water seepages were result of leakage from Dutard Reservoir; concluded that water in 2nd boring is due to occurrence of permeable lenses in the impermeable clays	Two borings, one upslope to 97 ft depth and one downslope of reservoir to 62 ft depth; upslope boring encountered no groundwater; downslope boring encountered water at 48 ft under confining pressure, stabilized at 12 ft.
Apr 1968 1	Gribaldo, Jacob, Jones, and Assoc.	Letter report to San Jose Water Works, Subject: Dutard Reservoir, Preliminary Stability Evaluation	Purpose to determine if a general condition of potential mass instability exists at the reservoir site (detailed investigation beyond scope of work)	No subsurface exploration done, but utilized results of borings from above study, with copies attached to this report

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Apr 1969 1	Leo M. Page	*Penitencia Tank Site and Proposed Treatment Plant Site*	Letter report to C.B. Church, describes ground failures following rains.	Hand drawn map showing areas of ground failure upslope and near SBA terminal reservoir.
May 1969 3, 4	Gribaldo, Jacob, Jones, and Assoc.	*Report to City of San Jose Dept. of Public Works, of Roadway Feasibility Investigation for Proposed Extension of Suncrest Drive, San Jose Highlands, San Jose, CA*	"...a major portion of the roadway will be underlain by a very large ancient landslide mass..."	Fifteen test borings drilled along proposed right-of-way, laboratory tests performed; boring logs and lab test results contained in report.
July 1971 2, 3, 4	Earth Sciences Associates	*Proposed Penitencia Creek (East San Jose) Water Treatment Plant* : Consultant's report prepared for James M. Montgomery, Consulting Engineers.	States "The site is located within and area of massive landsliding." and asserts their belief "that it would be a grave error to proceed with the design and construction of a … plant in the present location."	Letter report only, based on photogeologic investigation and on other reports in 1963 and 1964.
1971 1	U.S.G.S. Nilsen and Brabb	*Preliminary Photointerpretation and Damage Maps of Landslide and Other Surficial Deposits in Northeastern San Jose, California*	Interpretation of ground surface and origin of landforms; focus area of damages to developments included proposed treatment plant site, PCLS, and surrounding properties and streets; estimates of economic losses as consequence of development on landslide deposits.	Open file map prepared by viewing air photos w/ stereoscope; observations of damage made during 2 days of field checks in July & Sept 1971.
1972 4	U.S.G.S. Nilsen and Brabb	(Apparently same as above, but dated 1972 and designated "Miscellaneous Field Studies Map MF-361")	Geologic map, same as above	(See above)
Feb 1972 1	Kennedy Engineers	*Predesign Report, Penitencia Water Treatment Plant, Santa Clara County Flood Control and Water District*	States general findings of (post-dated) Dames and Moore Report (below)	N.A.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Mar 1972 1	Dames and Moore	*Soil and geologic studies, proposed Penitencia Water Treatment Plant, San Jose, California*: Consultant's report to the Santa Clara County Flood Control and Water District.	Asserts opinion that, from soils and engr. geol. standpoint, site is suitable for the planned plant, provided strict design and constr. criteria are followed; explicitly addresses ESA's 1971 letter and counters w/ opinion that site is not part of major active landslide; recommends "permanent precise survey network be established. . . monitored on a regular basis," also ". . . series of permanent water level observation wells be installed . . . could serve as an indication of developing instability . . "	Air photo interp. w/ stereo pairs; 3 test borings one to 330 ft, 2 to 100 ft; additional 8 test borings from 25 to 75 ft w/ lab testing. " Slotted plastic pipe with pea gravel backfill installed in six of the borings to observe groundwater conditions with time."

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
March 1972 1	Murray-McCormick Group	*Environmental Impact Analysis of the Penitencia Water Treatment Plant and Evergreen Pipeline*. Consultant's report for Santa Clara Valley Flood Control and Water District, San Jose, California.	References all of above studies, states some findings re landslide potential; has appendix containing copies of all reports except DWR and Gribaldo et.al. reports; "Our analysis indicates the need for additional in-depth studies to determine the geologic conditions of the site so that an assessment of risk and determination of impacts can be made." (pp 26) "The differences in opinion regarding the geologic conditions of the site have created a controversy. In part, this is due to abundant evidence of surficial instability and the lack of detailed data pertaining to the underlying geologic conditions." (pp 26) Additional discussion of geologic concerns on pp. 27-32. Reference to "...plans for reducing subsurface water have been prepared by the consulting engineer and geologist. ... will have a beneficial effect as regards the stability of the site." (Continued next column)	No exploratory borings installed (Cont'd) Referenced prepared plans, if in design criteria in Dames and Moore report, appear limited to 1) a layer of free draining granular backfill should be provided under water basins and behind basin walls "to minimize the chances of reduced stability due to a rise in the ground water table;" 2) "...subdrainage consisting of a granular blanket is provided to intercept leakage through the [washwater recovery] pond bottoms or rising ground water.," and 3) "...that surface run-off from paved areas be diverted into lined ditches and carried to a storm drainage system to be removed from the site."

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Apr 1972 1	E. Watson and T. Iwamura	Interoffice memorandum to Jack Watkins, “Review of Dames and Moore’s Soils and Geologic Studies - PWTP Site”	E. Watson, Water Supply Division Engineer and Tom Iwamura, Engineering Geologist, Water Supply Division, express concern and criticism of the report, citing unfavorable factors; handwritten notes found with this memo apparently from meeting “guess depth to base of slide mass at 247 ft elev.”	Recommendation “that at least five slope indicators and sealed piezometers be installed ASAP so that a maximum period of record could be acquired prior to construction.”
Jul 1972 1	Letter report from Geo-Testing, Inc. (Earl B. Hall, Director); proposal to install 6 inclinometers and 12 piezometers			
Aug 1972	Not indicated	*"Photographs of Existing Damage Caused by Soil Conditions in the Penitencia Treatment Plant Site Area"*	This is a collection in a comb binding of 84 3”x3” black and white photographs, each numbered, consecutively and described in 8 pages of text accompanied by a map showing the photograph locations identified by station number, of which there are 31.	Photographs are mainly of damage to curb and gutter, sidewalk, driveway, residential buildings, and roadway areas.
Sept 1972	Geo-Testing, Inc. installs 6 inclinometers (I-1 thru I-6) at Penitencia Water Treatment Plant site and 12 piezometers (P-1 thru P-12) in 6 piezometer borings, 1-PZ - 6-PZ, adjacent to inclinometers I-1 thru I-6. Boring logs and other drilling records not available. Geo-Testing, Inc. begins monthly inclinometer readings; District begins monthly piezometer readings.			
Spring 1973	Construction of Penitencia Water Treatment Plant begins.			
Apr 1974 3	Allied Geophysics and Frank Pisano & Associates	Lowder Residential Site, San Jose, California	Re parcel north of Dorel Dr., SW of SJWW storage and adjacent to PWTP site; “soft gouge” reported at depths of 12 to 18 feet underlain by shale	Two test borings
July 1974	Penitencia Water Treatment Plant begins operation.			

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Jun 1975	E. Watson and T. Iwamura	Letter report to Mr. David K. Gill responding to request for comments about inclinometer monitoring at PWTP	Provides 1) background of monitoring program, including purpose and description, data review procedures, role of Kennedy Engrs; 2) interpretation of data including "… very slow planar sliding motion was in progress at the site at the time of the inclinometer installation." 3) recommendations including "Insofar as detectable movements are occurring which could possibly have a future effect on plant operation, property damage, litigations, and public safety, stability monitoring has to be continued on a long-term basis."	
Sept 1975	Inclinometers Nos. 2A, 5A, and 6A installed to replace Nos. 2, 5, and 6 that were pinched at depth. Boring logs and other drilling information not available.			
Aug 1977	Geo-Testing, Inc., liquidated; purchase order for services assigned to Earl B. Hall, Inc.			
1978 3, 4	Judd Hull & Assoc.	*Soil and Geologic Investigation, Proposed 6-Lot Subdivision*; Consultant's report for Mr. Raymond Aiasa, San Jose, California	"...found that the eastern portion of a site located on Suncrest Avenue was 'underlain by the toe of a massive old landslide.' Found damage in sidewalk and curb adjacent to eastern part of site."	"No trenches were excavated on the site to evaluate the slide plane."
Oct 1978	Inclinometer I-3A installed to replace I-3 that had failed completely 4/78. Boring logs and other drilling information not available.			
1978 3, 4	Purcell, Rhoades, and Associates	*Geologic and Soils Investigations, Lots 38, 39, 40, of Sunset Hills Unit 3*	Study just east of Judd Hull study above. "No evidence of potential slide plane found."	Test borings apparently made, but of unknown depth.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Feb 1980 3	Bay Soils for Frank Pisano and Associates	*Geologic Investigation on Lands of Simpson, Dorel Drive, San Jose, California*	Parcel previously investigated in April 1974; "possible slide planes are indicated on logs of two of the four shallow borings."	Apparently four shallow borings.
Jul 1980 3, 4	Consulting Geologist Edward A. Danehy for Mr. Peter K. Chui	*Geologic Investigation of Bayo Vista Subdivision 1040 Nobel Avenue, San Jose, California*	Site located in toe area of slide mass; "In Trenches 1 and 5, there was clear evidence of a wet slippage plane and multicolored landslide material. Identical conditions were observed in Borings 1 and 2 at 24 to 28 feet and 20 to 24 feet respectively." Other evidence of landslide activity included resurveyed property lines and damage to man-made structures adjacent to the investigation site. "The slippage appears to be in small amounts and in episodes following above average rainfall seasons. Based on a 1980 resurvey of the property, up to three feet of westward movement has occurred in the eastern part of the property in the past 23 to32 years."	Five exploratory trenches excavated and two previously drilled test borings reviewed.
Oct 1980 1	Terrasearch for M & R Construction Company	*Soil and Geologic Investigation for a Proposed Residential Development, Grossmont Drive, San Jose, California*	A number of geotechnical reports reviewed; "Interpretation of test boring results supported the apparent conclusion that active landsliding was not a feature at the investigated parcel." Groundwater at 29 ft in one boring, localized	Reconnaissance mapping, five exploratory borings, two excavated test pits, laboratory testing; bottom of landslide mass defined at depths ranging from 12 to 44 ft "A large ancient landslide underlies the ...site."

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Dec 1980 3	E. A. Danehy	(title not known) supplemental geologic investigation due to additional concern for the potential existence of a deeper landslide plane at the site; supplements Danehy July 1980 investigation.	"... it was determined that a deeper landslide plane did not exist at the site."	Three additional borings.

<table>
<tr><th>Date
Notes</th><th>Author(s)</th><th>Title</th><th>Salient Information</th><th>Data Generated (e.g., from borings, inclinometers, etc.)</th></tr>
<tr><td>Feb 1982
1</td><td>Wahler Associates</td><td><i>Engineering and Geological Study, Penitencia Water Treatment Plant Expansion</i>, Consultant's report to Metcalf & Eddy, Inc.</td><td>Geotech study for proposed PWTP expansion. "...intended for planning purposes only"
This appears to be first study/report for SCVWD since EIA of 1972; gives excellent review summaries of a number of individual geologic and soils investigations for residential development in vicinity);contains good "Chronological development of investigations at Penitencia Water Treatment Plant Site" from 1962 through October 1980;
Acknowledges its adoption of use of term "Penitencia Creek Landslide" and cites references to same from geotechnical reports by others on studies in 1980 (Terrasearch, Inc., Danehy)
Salient conclusions: "...rate of movement cannot be predicted and may vary. Portions of the slide mass may not continue to move at the same rate or the slide may not continue to move as a unit."
Recommends additional studies, lab testing, additional instrumentation.</td><td>Scope of study limited to review of existing reports, data, etc., field reconnaissance of site to check for surficial signs of distress and instability, and analysis of info gained from those other sources.
"It is noted . . . the slide is suggested to be active at some locations and yet inactive in others."
Good summary of inclinometer data reported by Earl B. Hall to Jan 1982</td></tr>
<tr><td>May 1983</td><td colspan="4">Pipeline failures of SBA and 2 District pipelines by compression at toe of landslide. Repair work was completed in 1984 and included collapsible couplings on the three pipelines, and a vault at landslide toe for access to couplings, additional valving.</td></tr>
<tr><td>May 1984</td><td colspan="4">Last inclinometer measurements by Earl B. Hall, Inc. subsidiary of VHS Associates, Inc. before end of contract.</td></tr>
</table>

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Dec 1984 1	Coyle, J.M.	*Bedrock and Surficial Geology of the San Jose-Milpitas Area, Santa Clara County, California*; SJSU Master's thesis	Site mapped as active landslide	No subsurface exploration work performed. Extensive research of past investigations and studies and field mapping w/ geologic map and cross section passing through the PCLS
Jul - Aug 1985	Pitcher Drilling Co., for District, installs 6 inclinometers: I-10* thru I-15; *note gap in numbering (I-7 through I-9 not used) (Earl B. Hall, Inc. unable to bid on work due to District bonding requirements). Water Well Drillers Reports and boring logs are on file in District records.			
Nov 1985 1	R.L.Volpe & Associates for SCVWD	*Laboratory Investigation on Selected Samples from the Penitencia Creek Landslide, San Jose, California*	Presents results of lab investigation on eleven 3-inch dia. Pitcher barrel samples from six borings for inclinometer installations. Primarily clays with low to high plasticity, and some clayey sand.	Eleven samples from inclinometer hole nos. 10, 11, 12, 13, 14, 15, at depths of from 32' to 202'. Lab tests performed: sample description and classification (USCS); natural water content and dry density; grain-size distribution; Atterberg Limits; Specific Gravity; Direct Shear
Jul - Sept 1986	Pitcher Drilling Co., for District, installs 4 inclinometers I-16 thru I-19; and 14 piezometers PZ-13 thru PZ-26 in 8 piezometer borings, 7-PZ thru 14-PZ, adjacent to inclinometers. Water Well Drillers Reports and boring logs are on file in District records			
Mar 1987 1	Robert E. Tepel	*Background Information, Penitencia Creek Landslide*, Prepared for Board of Directors, SCVWD	An 8-page narrative describing District's work relating to the landslide	Summarizes preliminary landslide movement data (Table 1), and listing of data sources (Table 2).

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Apr 1987 1	R.L.Volpe & Associates for SCVWD	*Laboratory Investigation on Selected Samples from the Penitencia Creek Landslide, San Jose, California*	Laboratory test results from investigation of 35 samples collected during drilling of piezometer borings in Aug 1986.	Lab tests performed: Material description and classification; natural water content and dry density; specific gravity; grain-size distribution; Atterberg Limits; Permeability; Triaxial Shear Strength; also, some samples were examined by X-Ray Diffraction for clay mineral identification and by electron microscopy.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Dec 1987 1	Geothermal Surveys, Inc. for SCVWD	*Geohydrological Characteristics of the Penitencia Creek Landslide, Santa Clara County, California*	Collected geologic, geohydrologic and other information from previous investigations and existing data. Draws conclusions re strike and dip of slide surfaces; concludes that slide mass, with low permeabilities and vertical potential gradients, functions as an aquitard. A semi-confined aquifer may be present below the principal failure surface. Suggests that rapid piezometer response to rainfall may be caused by increased overburden pressure on the aquitard and resulting compression from shallow infiltration, not on recharge to the saturation range in the soil column. Presents 6 speculative working hypotheses with suggested tests re all possible sources of water entering the landslide. Appendix A is a good "History of Events" from early 1962 to Mar 1987	Conducted downhole temperature logging and determined thermal gradient in 19 existing boreholes to obtain information on water bearing formations within and beneath the slide. Report includes speculative maps of upper and lower potentiometric surfaces from piezometer data.
May 1991 1	Earth Systems Consultants for San Jose Water Company	*Supplemental Geotechnical Study, Dutard Heights Reservoir Tank Replacement*	Conducted in response to letter of concern from City of SJ; questions SCVWD inclinometer #16 readings	Test borings 4 and 5 to 48½ and 51½ feet, logs reported to refute presence of slide movement at 35 ft that is indicated by #16
Jun 1991	Pitcher Drilling Co., for SCVWD, installs 6 inclinometers, I-20 through I-25; original well permits and boring logs are on file in District records.			

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Nov 1991 1	Jo Crosby & Associates	*Environmental Impact Report on Suncrest, Appendix C, Geotechnical Data*	References 1972 investigation that included twenty-two boreholes and excavation of twenty geologic pits, with sampling and testing; 1979, two more boreholes on the site; 1988 and 1989 thirty-one additional bore holes drilled, logged and sampled on 36-acre parcel below the site; 1991, two 50-foot borings on upper and lower portion of site continuously sampled; Deep inclinometer hole drilled above PWTP - casing to 328 ft depth	Inclinometer readings, boring logs and trench logs "Presently, twenty-three geologic trenches have been excavated on the lower adjacent property and ninety-one on the investigated 52 acre parcel"
Jun 1993 1	VHS Associates for SCVWD	*Report of Instrumentation Services, Inclinometer Well Observations, Penitencia Water Treatment Plant, San Jose, California*	Summarizes measurements in upper accessible depths in six wells last read in May 1983; "Near surface movements of significant magnitude have occurred at all locations except Well 6A during the past 10 years." Other pertinent conclusions reached.	Data summarized from inclinometer well nos. 1, 2A, 3A, 4, 5A, and 6A

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Aug. 9, 1993 1	Woodward-Clyde Consultants	*Technical Memorandum, Penitencia Water Treatment Plant Site Study*, prepared for Camp Dresser & McKee, Inc. This study was authorized by Camp Dresser & McKee, Inc. (CDM) in support of Phase 2B of the Water Quality Regulation Compliance Project (WQRCP) for the SCVWD. This is a large report containing 94 figures, 9 tables, and six appendices in three volumes in addition to the main text volume. This report is later referred to as the PWTP Phase 2A Site Study report	Presents results of a geotechnical study to assess potential risk of earthquake-induced landslide movement beneath the Penitencia Water Treatment Plant. "The PWTP property has experienced, and continues to experience, movement in the downhill direction. ... the PWTP is situated on a large, deep-seated landslide. The toe ... is fairly well defined ... what remains unclear are the uphill limits ... commonly referred to as the head scarp. ... We estimate an earthquake of magnitude 7 on the Hayward fault could trigger landslide movements in the range of 1 ft to 6 ft." (Note: Although a probabilistic seismic hazard analysis was performed for the site, the potential landslide deformation was based on a MCE of 7.0 and a peak ground acceleration (PGA) of 0.62g.)	(No subsurface exploratory work was performed for this study.) Reviewed data from 42 inclinometers, 24 piezometers and land surveys by SCVWD and DWR. (Some discrepancies are evident in tabulated piezometer data.) Performed limit equilibrium slope stability analyses; downhole geophysical (velocity) measurements in two inclinometer casings; dynamic modeling of landslide mass using non-linear finite-element analyses; evaluated yield acceleration from past earthquakes to estimate potential deformation for future earthquake on Hayward fault; estimated potential range of earthquake-induced deformation for an earthquake of magnitude 7 on Hayward fault. Estimates derived from finite element analysis result in potential landslide movement of about 2 feet from a M_w 7 .
Aug. 10, 1993	City of San Jose adopts an Urgency Ordinance imposing a six-month moratorium on all development approvals in the "East Foothills Area"			

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Feb 1994 1	Woodward-Clyde Consultants	*Penitencia Water Treatment Plant Geotechnical Investigation*, for CDM (This report is found in the Water Quality Regulation Compliance Project Engineer's Report, Volume 8, Geotechnical Investigations and Summary Report, September 5, 1997, by CDM for SCVWD.)	This investigation was performed concurrently with above study; objective was to provide preliminary opinions and recommendations regarding the geotechnical engineering aspects of design of treatment facilities expansion.	Subsurface investigation in vicinity of proposed improvements, including six borings, all to a depth of 44.5 ft; two backhoe pits excavated to 3 and 3.5 ft depths. Testing of borehole samples collected at approx. 5-ft intervals in each boring, included water content, dry density, unconfined compressive strength; Atterberg Limits determined on 2 samples.
Mar 1994 1	Geologic Review Panel for City of San Jose Dept. of Public Works	*San Jose East Foothills Moratorium Area Geologic Review Panel Report*	Panel "convened by City to respond to new data and information ... contained in the [WCC] PWTP report." Panel questions WCC's identification of a scarp on hillslope east of PWTP site, and asserts that "additional field work would be required to establish the actual upper extent of this landslide" (Ref. p. 15). Recommendation made "...that the City modify its review requirements for proposed developments projects within the Moratorium Area to reflect the latest information in this geologically complex and potentially hazardous area."	No subsurface exploration conducted. Study included review and evaluation of 1993 WCC report and a number of previous geologic/geotechnical reports, field reconnaissance and review of aerial photos, and providing additional insight as to the limits of the PCLS. Recommendations include six specific areas that should be addressed with regard to requirements placed on applicants and the City as preliminary, submittal, and review criteria for the approval of any given development proposal.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Aug 31, 1995 1	Woodward-Clyde Consultants	*Technical Memorandum - Phase 2B, Tasks 14.2.1, 14.2.2, 14.2.3, Penitencia Water Treatment Plant Landslide Evaluation, San Jose, California*, for CDM for SCVWD's WQRCP	Whereas the Phase 2A Site Study report analysis was based on a MCE of M_w = 7 on the southern segment of the Hayward fault, the analyses in this report are based on ground motions determined to have a 10% probability of being exceeded in the next 50 years, and hence a PGA = 0.70g, and for different groundwater conditions. Unlikely that lowering groundwater level would effectively reduce magnitude or risk of landslide-induced deformation.	Different groundwater (high, normal, low) and ground motion (PGA=0.62 and 0 .70) conditions considered: With highest groundwater and ground motion conditions, estimates from finite element analysis are that landslide movement could approach 4 feet; 2 ft and 3 ft from 0.62 and 0.70 PGAs under other groundwater conditions. Long-term creep displacement also evaluated.
Oct 1995	Norfleet Consultants, Earth Mechanics Consulting Engineers, and Philip A. Frame, Consulting Engineering Geologist	*Final Report, Phase IA Regional Geologic Study of the Special Geologic Hazard Study Area for The City of San Jose Department of Public Works*	The special study area is in the east foothills of San Jose just north of Alum Rock Park. "It involved a broad review of existing technical reports, historical aerial photographic review, geologic and geomorphic mapping, inventory of historical damage, geophysical review, and engineering analyses. A key aspect . . . was that the geologic and engineering conditions were evaluated on a regional basis."	Re seismic stability of the landslide, this report confirmed by independent analyses that deformation due to a MCE on the Hayward fault would range from 1 to 6 feet. "Even though this study suggests that it is unlikely that classical Hayward Fault exists where is (sic) has been previously mapped in the Study Area, it was modeled to evaluate WCC procedures and to provide recommendations for future analyses."

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Jan 1997 1	Woodward-Clyde Consultants	*Supplementary Geotechnical Review and Recommendations, SCVWD Water Quality Regulation Compliance Project, Santa Clara County, California*, for Camp Dresser & McKee, Inc.	Provides updated geotechnical engineering design criteria based on decision to move forward with Option 2, Settled Water Ozonation. Includes the following: "Evidence of secondary slide planes within the active landslide has been observed in some of the inclinometers. However, interpretation of the secondary slide planes was beyond WCC's current scope of study." (See add'l information in next column.)	Recommends two additional exploratory borings to 50 ft depth, and one observation well to 35 ft depth. "As discussed in Task 5 Report...WCC recommended a minimum of one inclinometer be installed within each of the proposed expansion areas. The inclinometers should be monitored for at least two years. This will provide further insight regarding the presence and activity of possible secondary zones of sliding, as well as further confirmation of the rate and direction of the deep-seated landslide movement."
Aug 1997	Pitcher Drilling, for District, installs 2 inclinometers: I-26 and I-27, as recommended/referenced in WCC's Jan 1997 report above. Boring logs in WCC report and originals in District files.			

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Dec 1997 1	Woodward-Clyde Consultants	*Inclinometer Installations I-26 and I-27, Penitencia Water Treatment Plant, San Jose, California, Santa Clara Valley Water District Project 932307* Letter report to Mr. Robert Tepel, Engineering Geologist, SCVWD	Sample collection during the drilling was strategically designed to collect continuous samples, in 3-ft Shelby tubes with Pitcher barrel sampler, in I-26 from 175 to 195 feet deep, and in I-27 from 150 to 171 feet below ground surface. It is believed that the sample collected from I-27 probably includes the primary slip surface between elevation 270 and 271 ft, inferred from striated slip surface observed in the laboratory. Bulk samples were obtained from rotary wash drill cuttings, and indication of secondary slide surface would have been mostly impossible to distinguish by that method.	Boring logs of I-26, depth of 230 ft (elev. = 207 ft), I-27 to depth of 210.5 ft (elev. = 224.5 ft) Depth intervals for continuous sampling were targeted to locate the primary slip surface suggested by other nearby inclinometer records. Lab testing included moisture content and dry density, unconfined compressive strength, plasticity index, and grain size distribution including hydrometer testing, all on selected samples.

Date Notes	**Author(s)**	**Title**	**Salient Information**	**Data Generated** (e.g., from borings, inclinometers, etc.)
Aug 2000 1	Harza Engineering Co., Inc.	*Geotechnical Investigation, SCVWD Water Treatment Improvement Project - Stage 2, Penitencia Water Treatment Plant (WTP) San Jose, California*	(Still in draft form as of this writing.) "The purpose of our investigation was to evaluate the foundation soils and provide recommendations concerning the geotechnical engineering aspects of the project."	Three new borings drilled and two CPTs advanced. Two borings to depth of 50 ft near CPT to 65 ft (at washwater site); one boring to 40 ft near CPT to 75 ft (near toe of slide piping modification). Lab tests included natural water content, dry density, Atterberg Limits, percent passing the #200 sieve, unconfined compression tests, and "R" value tests, all on selected samples.

APPENDIX B
DETAILED PIEZOMETER RESULTS

1.0 INTRODUCTION

This appendix presents details regarding the 26 piezometers installed for or by the District at the Penitencia Creek Landslide (PCLS). It includes information deemed most relevant to the collection and qualification of the data derived from the piezometer measurements. The piezometer measurements are converted directly to piezometric surface elevations (PSEs) for interpretation regarding ground water conditions.

The PSE, as an engineering quantity for geotechnical engineering purposes, is derived from the Bernoulli equation, which is defined as follows:

Total head = Pressure head + Elevation head + Velocity head

For problems involving flow through porous media, such as ground water flow through natural slopes, the velocity head is very small (<< 1 foot) and can therefore be safely neglected. Pressure head is read from a piezometer as pore water pressure at the piezometer sensing tip, as measured (for example) in pounds per square inch (psi), which is converted to feet of pressure head by multiplying it by 2.31 (= 144 sq in. per sq ft / 62.4 lbs per cubic foot). The Bernoulli equation therefore reduces to the following:

Piezometric Surface Elevation = Pressure head + Elevation Head

where:

Pressure head (ft) = piezometer reading in psi x 2.31
Elevation head (ft) = piezometer tip elevation in ft.

A plan view showing the location of the piezometers is presented in Figure B-1. Typical installation details are shown in Figure B-2. Piezometers are numbered P-1 through P-26 and have been installed in two distinct series: the first series of 12 was installed in 1972, and the second series of 14 was installed in 1986. All piezometers are of the pneumatic type and were installed in 14 borings, numbered 1-PZ through 14-PZ. In all but the two borings 7-PZ and 13-PZ, two piezometers were installed in each boring at carefully selected depths below the ground surface. There has been some confusion in the past due to the unfortunate selection of "PZ" used to define the drill hole number. For those not familiar with the numbering scheme, it was easy to confuse the drill hole number and piezometer number with each other. Table B-1 presents a summary of installation particulars for all piezometers. Table B-2 contains parameters necessary for converting raw measurement numbers into useful information for interpretation according to the relationships and equations described above.

A key feature of the historical record keeping of piezometer information has been the development and implementation of the Piezometer Data Sheet (District form number FC 613).

Use of that form was implemented in 1980 to have all pertinent information surrounding a given piezometer contained on a single sheet. These data sheets have been updated with annotations, and copies are included in this appendix for future reference.

All piezometer measurements since the first installations in 1972, and converted to PSEs, are presented graphically in Figures B-3 through B-16. Two cross sections were developed that show the ground profile, piezometer location, and resulting piezometric surface elevations for selected piezometers that could be reasonably projected to the planes represented by the vertical sections. (Figures B-18 and B-19). These cross sections are further discussed in Section 3.0 of this appendix.

2.0 PIEZOMETERS

2.1 Hall Piezometers

Geo-Testing Company, Inc. installed the first 12 piezometers under the supervision of the company's director Earl B. Hall, in September 1972. The piezometers were installed in six borings, two piezometers per boring, adjacent to and concurrently with the first six inclinometers (also installed by Geo-Testing Co.). These first piezometers are Hall Hydrostatic Pressure Cells, Model HPC-7 (a copy of the specification sheet is included in this appendix). Documents describing the piezometers' installation include two letter reports from Geo-Testing Company, Inc., dated 14 July 1972 and 14 November 1972, and the referenced attachments that include a manufacturer's calibration curve for each piezometer and a location map. Copies of those letters and the referenced attachments are included at the end of this appendix. A search for boring logs of the instrument installations was unsuccessful.

Piezometer measurements are taken at the piezometer boring location in the field using a specially designed pneumatic pressure indicator. Geo-Testing Company took the initial readings of its 12 piezometers on 13 September 1972 following installation. Since that time, District staff have taken the measurements monthly until mid 1974, then monthly to quarterly until mid 1977, and quarterly thereafter until mid 1987 when readings of the Hall piezometers were discontinued. Readings were taken once in 1992 and then again in 1995. Quarterly readings were resumed in 1996, but then discontinued after the December 1999 readings due to reported malfunctioning of the pneumatic pressure indicator console. Quarterly readings have since resumed with the October 2000 readings following repair of the indicator console. Exceptions to the above reading schedules have been piezometers P-1 and P-2, for which December 1982 was the last reading due to water in the pneumatic tubing that connects to the indicator console. Piezometer P-1 was then read sporadically for a total of 6 readings from 1992 through 1997.

During compilation of data for this appendix, it was noted that piezometer data had not been consistently compiled over the years. Prior to 1986 and for any given reading, the reported value was the depth from the ground surface to the piezometric surface, and it was computed by incorporating calibration factors provided by the instrument manufacturer. Since 1986 and until this report, the piezometric surface has been reported as a PSE (as distinct from the depth to the PSE), and without the calibration factor applied. The PSEs were computed based on elevations estimated from topographic maps, even though District surveyors had determined more exact elevations in 1992. For this report therefore, all piezometer data have been recalculated using the original calibration factors applied to the Hall piezometers as appropriate, and using the surveyed ground surface elevations for all instruments producing data. The datum for the surveyed ground surface elevations used and the resulting PSEs is NGVD 1929. This distinction should be kept clear, as the topographic contours and elevations referenced for all current work at the site are based on NAVD 1988.

2.2 SINCO Piezometers

The Slope Indicator Company (SINCO piezometers) manufactured the second series of 14 piezometers. The District installed the 14 SINCO piezometers, numbered P-13 through P-26, in July and August of 1986, also adjacent to inclinometers, to better define the variation and distribution of the pore pressures acting within the landslide. The SINCO piezometers were installed in eight borings numbered 7-PZ through 14-PZ, with two piezometers per boring, except for a single piezometer in boring 7-PZ, at carefully selected depths. Pitcher Drilling Company advanced the borings and performed the piezometer construction for the District. District staff recorded boring logs and documented the installation in accordance with: "District Specifications and Contract Documents for the Installation of Three Inclinometer and Twelve Piezometer Systems at Penitencia Water Treatment Plant, Project No. 9323, May 1986."

The SINCO piezometers were read monthly following installation and through March 1988. They have since been read quarterly to the present, except during the period from January 1991 through August 1992, during which no readings were taken. All SINCO piezometer measurement data have been recalculated for this report using the surveyed ground surface elevations. The Piezometer Data Sheets for P-13 through P-26 also include annotations reflecting the updated reference elevations and resulting change in the respective equations used to calculate the PSEs (see discussion above pertaining to reference elevations and recalculations of PSEs).

3.0 PIEZOMETER GRAPHS

As an aid to interpreting the individual piezometer results, each of the Figures B-3 through B-16 has a bar chart type of graph at the bottom of the page showing monthly rainfall (in inches) from July 1971 through March 2000. Using a consistent horizontal time axis, piezometer measurement data appears on a larger graph above the rainfall graph. Each piezometer graph corresponds to a unique piezometer boring. Most of the figures show two sets of data plotted on each graph; they show data for the two piezometers installed in the same boring. Borings 7-PZ and 13-PZ each contain only one piezometer, P-13 and P-24, respectively. It will be noted in each figure that the tip elevations are stated in the legend and shown as a solid or dashed horizontal line on the graph, the sensing intervals are stated in the legend. The ground surface elevation is stated in the lower right hand part of the page along with the piezometer and boring identification and installation information.

An attempt has been made to maintain consistency in graphical presentation with respect to the Piezometric Surface Elevation (the vertical axes). The horizontal scale is uniform among all time series plots, from January 1971 through December 2000, regardless of when the piezometer was installed. The vertical axes (PSE) on all piezometer measurement graphs, with several exceptions, span a distance of 100 feet elevation. The exceptions are the graphs in Figures B-8, B-12, B-14, and B-16, which span 200 feet vertically. We believe that this approach provides a consistency of scale for more direct comparison of groundwater measurement trends.

It will be noted that other data points are shown on the graphs for Hall piezometers (Figures B-3 through B-8). Those points are from information gathered beginning in late 1976, and represent the free water surface acting within the casings in adjacent inclinometers as noted during semi-annual inclinometer readings. Those points have been defined in the legends as being observation well readings. It should be noted that it is not known where the water is entering the casing and it would be erroneous to assume that the indicated water level represents a free water surface.

2.4 CROSS SECTIONS

The cross-sections shown in Figures B-18 and B-19 have been developed as additional tools for interpreting the PSEs measured during the period of record. It should be noted that due to the distance over which piezometers are located at the site that it was necessary to use a vertical exaggeration of five to one (5:1) in order to minimize the size of the figure. The piezometer data shown in the referenced figures represent the arithmetic average PSEs over the historical period of record for the Hall piezometers, and since 1992 for the SINCO piezometers, with spurious data points removed for more representative information. When viewing the PSE data in this manner, it was quickly discovered that it was also necessary to know the location of the failure plane and the proximity of a given piezometer to the failure plane. The location of other principal structures at the site, namely the PWTP Clearwell and DWR Terminal Reservoir, are also shown where they cross the section.

Three different piezometric surfaces have been interpreted from the data. A detailed discussion of these piezometric surfaces is presented in Section 6.6.

Table B-1
Summary of Piezometer Installation Data

Piezometer Boring No.	Installation Date	Adjacent Inclinometer(s)[1]	Piezometer Number, Type[2]	Depth (ft) to Tip	Depth (ft) of Sensing Zone
1-PZ	Sept. 1972	I-1, I-12	P-1, h	69.0	67.5 – 71.5
			P-2, h	40.0	39.5 – 41.0
2-PZ	Sept. 1972	I-2, I-2A	P-3, h	70.0	68.5 - 72.0
			P-4, h	40.0	38.0 - 42.0
3-PZ	Sept. 1972	I-3, I-3A	P-5, h	70.0	68.5 - 72.0
			P-6, h	38.5	37.0 - 39.5
4-PZ	Sept. 1972	I-4, I-14	P-7, h	70.0	68.5 - 72.0
			P-8, h	39.0	38.5 - 40.0
5-PZ	Sept. 1972	I-5, I-5A	P-9, h	70.0	68.5 - 72.0
			P-10, h	40.0	38.5 - 41.0
6-PZ	Sept. 1972	I-6, I-6A	P-11, h	70.0	68.5 - 72.0
			P-12, h	40.0	38.5 - 41.0
7-PZ	Aug. 1986	I-10	P-13, s	45.0	39.0 - 45.0
8-PZ	Aug. 1986	I-11	P-14, s	125.0	123.0 - 126.0
			P-15, s	90.0	85.0 - 90.0
9-PZ	Aug. 1986	I-13	P-16, s	144.0	138.0 - 144.0
			P-17, s	80.0	75.0 - 82.0
10-PZ	Aug. 1986	I-4, I-14	P-18, s	195.5	192.0 - 195.5
			P-19, s	145.0	137.0 - 148.0
11-PZ	Aug. 1986	I-17	P-20, s	115.0	112.0 - 116. 5
			P-21, s	87.0	83.0 - 92.0
12-PZ	Jul. 1986	I-16	P-22, s	230.0	226.0 - 231.5
			P-23, s	135.0	132.0 - 141.0
13-PZ	Aug. 1986	I-18	P-24, s	145.0	142.0 - 148.0
14-PZ	Aug. 1986	I-19	P-25, s	230.0	226.0 - 231.0
			P-26, s	134.0	130.5 - 134.0

1. Inclinometers I-1 through I-6 (without “A” suffix) were installed concurrently with the Hall piezometers; I-10 Through I-19 were installed in 1985 or later. (There is a gap in inclinometer numbering; numbers I-7 through I-9 have not been used.)

2. h = Earl B. Hall Hydrostatic Pressure Cell, Model HPC-7, read with Geo-Testing, Inc. pneumatic pressure indicator.
   s = Slope Indicator Company (SINCO) pneumatic cell, Model 514177, read with SINCO pneumatic pressure indicator.

Table B-2
Summary of Piezometer Measurement Parameters

Piezometer Number	Orig. Estimated Ground Surface Elev. (ft)	Surveyed Ground Surface Elev. (NGVD 1929)	Measured Depth (ft) to Tip	Tip Elev.	Sensing Interval	Equation Used to Determine Piezometric Surface Elevation (PSE) (All elevations NGVD 1929 datum) (Readings are in psi)
P-1	402.0	401.09	69.0	332.09	329.6 – 333.6	PSE = 332.1 + 2.31 x (Reading - 1.05)
P-2	402.0	401.09	40.0	361.09	360.1 – 361.6	PSE = 361.1 + 2.31 x (Reading - 1.20)
P-3	351.0	349.50	70.0	279.50	277.5 – 281.0	PSE = 279.5 + 2.31 x (Reading - 1.35)
P-4	351.0	349.50	40.0	309.50	307.5 – 311.5	PSE = 309.5 + 2.31 x (Reading - 1.06)
P-5	453.0	452.14	70.0	382.14	380.1 – 383.6	PSE = 382.1 + 2.31 x (Reading - 1.50)
P-6	453.0	452.14	38.5	413.64	412.6 – 415.1	PSE = 413.6 + 2.31 x (Reading - 1.28)
P-7	442.0	442.56	70.0	372.56	370.6 – 374.1	PSE = 372.6 + 2.31 x (Reading - 0.55)
P-8	442.0	442.56	39.0	403.56	402.6 – 404.1	PSE = 403.6 + 2.31 x (Reading - 1.34)
P-9	345.0	345.30	70.0	275.30	273.3 – 276.8	PSE = 275.3 + 2.31 x (Reading - 0.70)
P-10	345.0	345.30	40.0	305.30	304.3 – 306.8	PSE = 305.3 + 2.31 x (Reading - 0.70)
P-11	359.0	358.99	70.0	288.99	287.0 – 290.5	PSE = 289.0 + 2.31 x (Reading - 1.43)
P-12	359.0	358.99	40.0	318.99	318.0 – 320.5	PSE = 319.0 + 2.31 x (Reading - 1.23)
P-13	284.0	284.46	45.0	239.46	239.5 – 245.5	PSE = 239.5 + 2.31 x Reading
P-14	375.0	375.66	125.0	250.66	249.7 – 252.7	PSE = 250.7 + 2.31 x Reading
P-15	375.0	375.66	90.0	285.66	285.7 – 290.7	PSE = 285.7 + 2.31 x Reading
P-16	389.0	390.13	144.0	246.13	246.1 – 252.1	PSE = 246.1 + 2.31 x Reading
P-17	389.0	390.13	80.0	310.13	308.1 – 315.1	PSE = 310.1 + 2.31 x Reading
P-18	443.0	443.47	195.5	247.97	248.0 – 251.5	PSE = 248.0 + 2.31 x Reading
P-19	443.0	443.47	145.0	298.47	295.5 – 306.5	PSE = 298.5 + 2.31 x Reading
P-20	449.0	449.26	115.0	334.26	332.8 – 337.3	PSE = 334.3 + 2.31 x Reading
P-21	449.0	449.26	87.0	362.26	357.3 – 366.3	PSE = 362.3 + 2.31 x Reading
P-22	536.0	535.23	230.0	305.23	303.7 – 309.2	PSE = 305.2 + 2.31 x Reading
P-23	536.0	535.23	135.0	400.23	394.2 – 403.2	PSE = 400.2 + 2.31 x Reading
P-24	414.0	414.64	145.0	269.64	266.6 – 272.6	PSE = 269.6 + 2.31 x Reading
P-25	552.0	549.85	230.0	319.85	318.9 – 323.9	PSE = 319.9 + 2.31 x Reading
P-26	552.0	549.85	134.0	415.85	415.9 – 419.4	PSE = 415.9 + 2.31 x Reading

T 1
R A

GEO-TESTING, INC. 7

~~P. O. BOX 959 SAN RAFAEL, CALIFORNIA 94902~~ (AREA CODE 415) 472-2100
P. O. Box 4339 San Rafael, California 94903
- *EARTH MATERIALS TESTING*
- *INSTRUMENTATION*
- *RESEARCH*

14 July 1972

Santa Clara County
Flood Control and Water District
5750 Almaden Expressway
San Jose, California 95118

Attention: Messrs. Jack Watkins, Jr. and T. I. Iwamura

Subject: Instrumentation - Penitencia Water Treatment Plant

Gentlemen:

Reference is made to the writer's visit to your offices on 6 July 1972 and the following visit to the subject plant site with your Mr. Iwamura. Our understanding of the project is as follows.

Twelve holes are to be drilled varying from 70 feet for the piezometer holes to a depth of 250 feet for the inclinometer holes. Six inclinometer holes will be drilled and six piezometer holes will be drilled. Each piezometer hole is to contain two piezometers, one at a depth of approximately 70 feet, and the other at a depth of approximately 40 feet. The locations for these installations are outlined on a drawing by Kennedy Engineers entitled Site Grading, Sheet 3 of 8.

Further, it is our understanding that the District will stake out the locations of each site and that if any leveling is necessary to accommodate a drill rig the District will level a pad sufficiently to accommodate the rig.

It is our suggestion that at each location where instruments will be installed the piezometer hole will be five feet from the inclinometer hole.

We propose to drill six inclinometer holes totaling 950 feet, and six piezometer holes with a total depth of 420 feet. Each inclinometer hole will contain Hall Inclo-Meter casing which will be grouted in place. The six piezometer holes will each contain two Hall Hydrostatic Pressure Cells at approximate depths of 70 feet and 40 feet and will terminate at a small terminal plate at the top of the hole in a steel riser pipe. Both the inclinometer holes and the piezometer holes will contain vandal-proof terminal caps that will accommodate a padlock. We are assuming that the padlocks will be furnished by the District.

Also, around each inclinometer hole and piezometer hole will be a 30-inch square concrete pad approximately four inches thick at the edge and approximately eighteen inches thick around the terminal pipe. This design will serve two purposes; one, it will make it extremely more difficult for vandals to damage the terminal pipes, and two, it presents a small but adequate working platform for the technicians to stand on during inclement weather when taking subsequent observations.

As indicated before, the aluminum inclinometer casing will be grouted in place. The Hall Hydrostatic Pressure Cells in the piezometer holes will be sealed with a special bentonitic seal between the two piezometers and a seal will be placed a minimum depth of five feet above the top piezometer. The remaining portion of the hole will be filled with cuttings that were removed from the hole during the drilling operation. The two Hall Hydrostatic Pressure Cells that are proposed in each of the piezometer holes are of the same type that was installed in Guadalupe Dam, and the console that the District purchased for the Guadalupe structure can also be used for monitoring these units.

All inclinometer holes will be observed after installation, and an initial or reference baseline will be established for each hole. Approximately 30, 60 and 90 days after the initial installation, complete observations will be made of the six inclinometer holes, and any movements will be reported on a profile plot together with a letter report.

All of the services and materials as outlined above, including the three observations after completion of the project, will be furnished at a cost of $28,260.00. If additional observations are desired beyond those listed herein, a charge of $1,350.00 per observation will be made. This cost will include the apparatus rental costs as well as the data reduction, in-house engineering and letter report.

At the present writing, it would appear that we can begin drilling and installation operations within approximately 15 days of receipt of notice to proceed.

Based on the conditions observed at the site, we would strongly recommend that you use the Hall Hydrostatic Pressure Cell for observing the piezometric pressures as opposed to the porous piezometer tubes that are referred to in our alternate. It is our belief that the imperviousness of the area will cause a significant time lag in the observation tube of the porous piezometer, whereas the Hall Hydrostatic Pressure Cell is a no-volume-change unit and will respond to pore pressure changes immediately.

Your Mr. Iwamura had asked the writer to furnish a cost estimate for the complete inclinometer monitoring system. At the present moment one of our principal suppliers for the Teletype unit has moved to the Chicago area, and we are attempting to locate a local source that will meet our specifications. Therefore it is anticipated that within the next 30 days we will have a proposal to you for a complete inclinometer monitoring system using the Hall Inclo-Meter and cable, console and Teletype with punch tape.

We appreciate the opportunity of presenting this proposal to you, and we will look forward to an early reply.

Yours very truly,

GEO-TESTING, INC.

Earl B. Hall

Earl B. Hall
Director

EBH:cg

cc
JW w/A
RA

GEO-TESTING, INC.

~~P.O. BOX 959 SAN RAFAEL, CALIFORNIA 9490[illegible]~~ (AREA CODE 415) 472-2100
P. O. Box 4339 San Rafael, California 94903

- EARTH MATERIALS TESTING
- INSTRUMENTATION
- RESEARCH

11-15

14 November 1972

Santa Clara County
Flood Control and Water District
5750 Almaden Expressway
San Jose, California 95118

Attention: Mr. J. Watkins, Jr. & Mr. T. I. Iwamura

Subject: Piezometer Installation
Penitencia Water Treatment Plant

Gentlemen:

With this letter we transmit the calibration curves for the 12 Hall Hydrostatic Pressure Cells installed during September 1972 at the subject site. The hydrostatic pressure cells were installed at the approximate locations shown on the attached plot plan, Plate 1. These are adjacent to the inclinometer casings that were installed about the same time. The installation was performed according to the procedures described in our 14 July 1972 letter.

The below tabulation shows the depths of the tips of the hydrostatic pressure cells, the sensing zones (top and bottom of sand backfill between bottom of boring or bentonite seals) and the readings that we made 13 September 1972.

Piezometer	Hole	Tip Depth (Feet)	Depth of Sensing Zone (Feet)	13 Sept. '72 Reading (psi)
P-1	1	69	67½ to 71½	14.2
P-2	1	40	39½ to 41	12.2
P-3	2	70	68½ to 72	10.8
P-4	2	40	38 to 42	1.1
P-5	3	70	68½ to 72	20.8
P-6	3	38½	37 to 39½	8.5
P-7	4	70	68½ to 72	15.7

P-8	4	39	38½ to 40	5.8
P-9	5	70	68½ to 72	9.2
P-10	5	40	38½ to 41	0.9
P-11	6	70	68½ to 72	22.2
P-12	6	40	38½ to 41	7.0

We understand that you will continue the piezometer readings using the console from the Guadalupe Dam. The readings should be taken with a flowmeter setting of 6.5 ± 1.5.

Yours very truly,

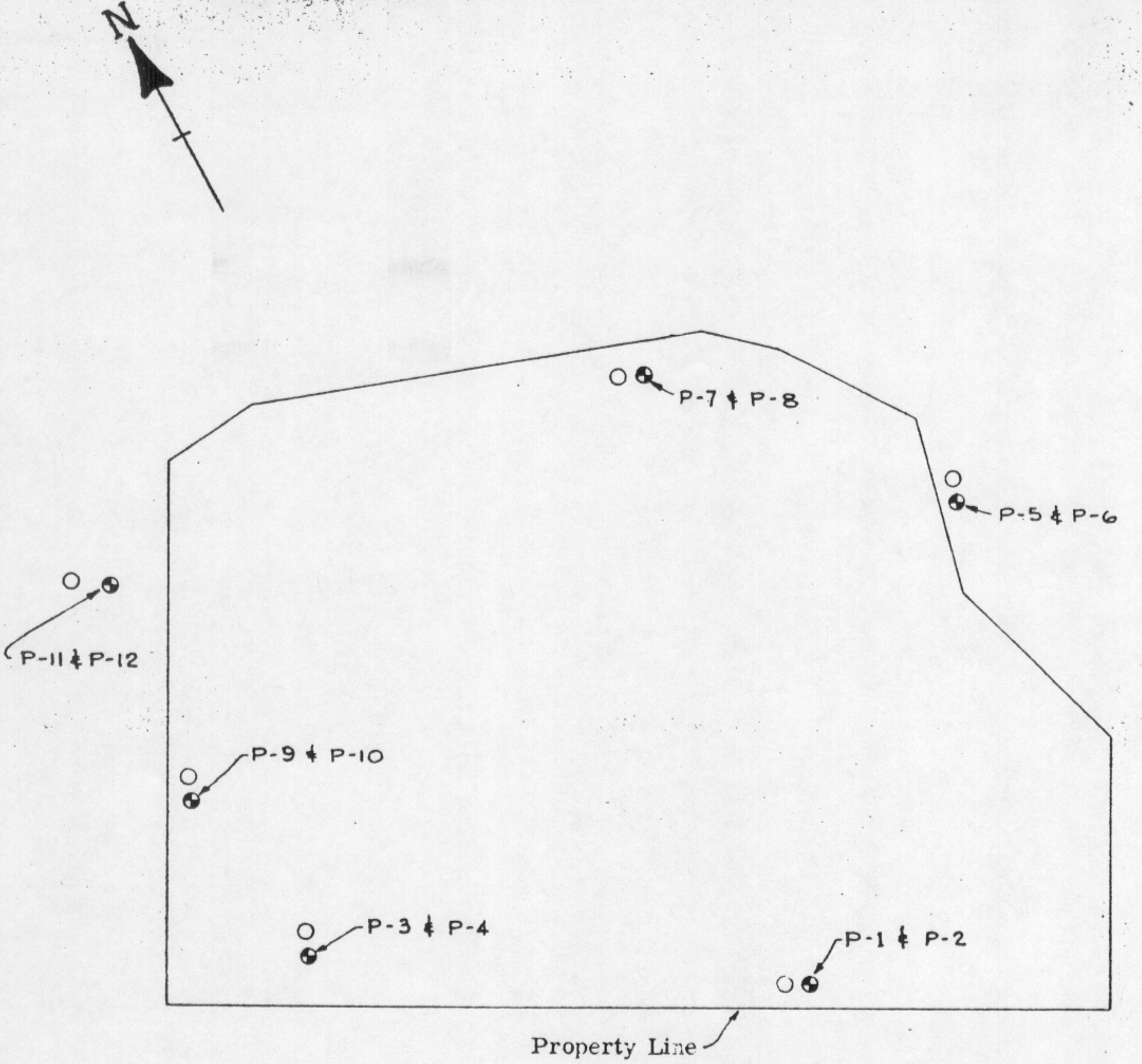
GEO-TESTING, INC.

Earl B Hall

Earl B. Hall
Director

EBH:RAB:mc

Enclosures



LEGEND

○ Inclinometer Casing
● Pressure Cell Boring

Location Plan

Hall Hydrostatic Pressure Cells

For

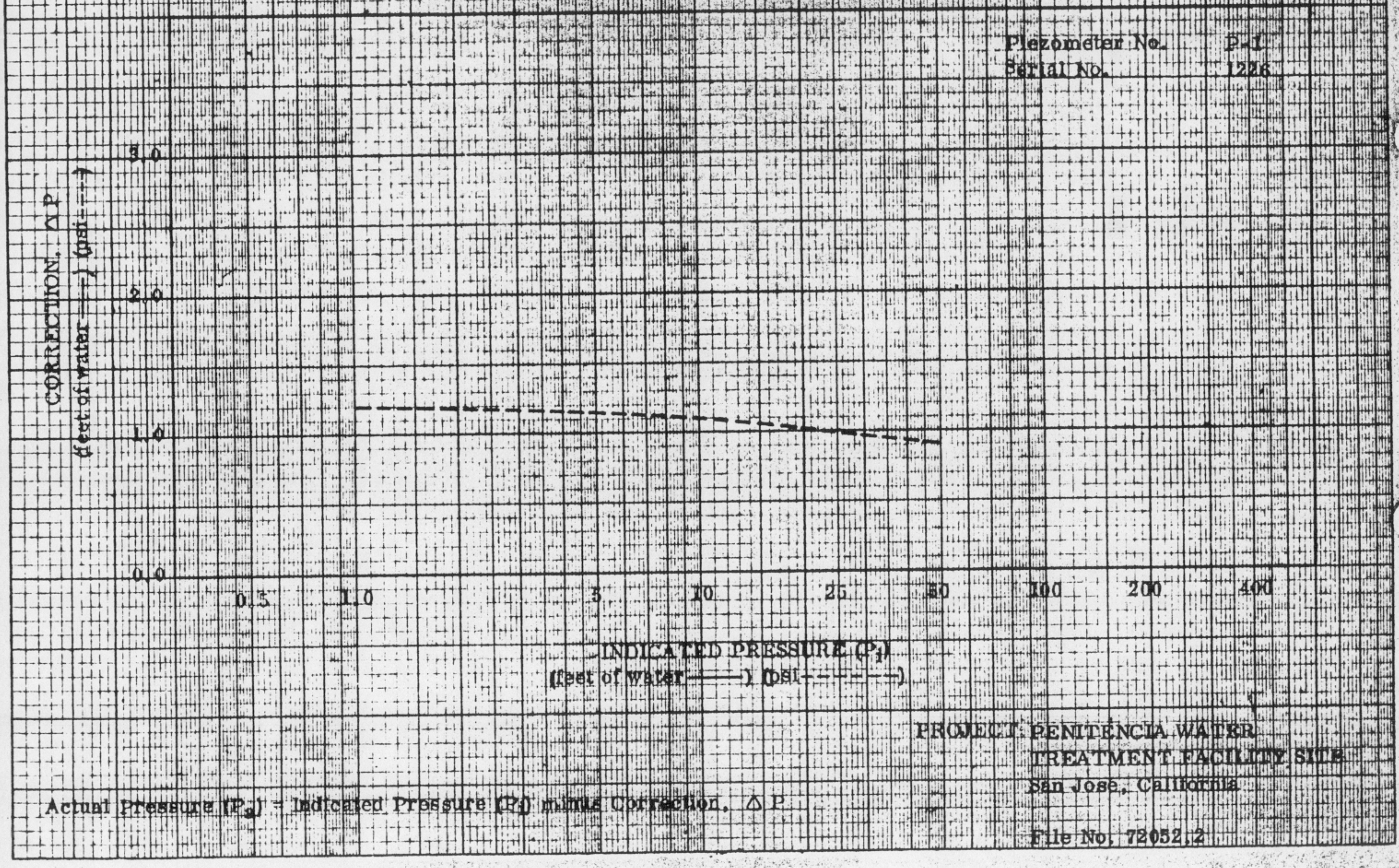
Penitencia Water

Treatment Facility Site

San Jose, California

GEO-TESTING, INC.
SAN RAFAEL
CALIFORNIA

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA



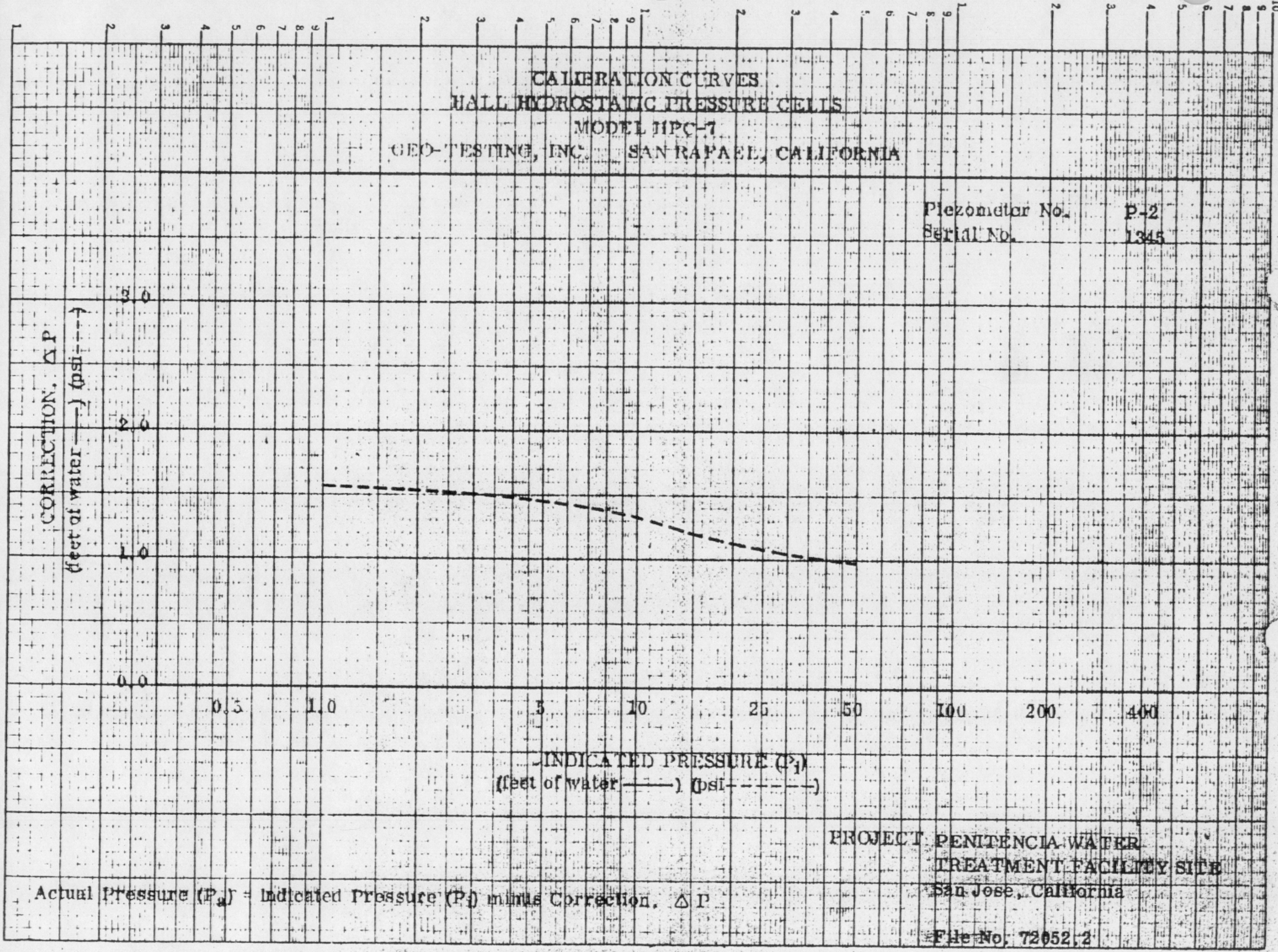
PROJECT: PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, △ P

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA

Piezometer No. P-2
Serial No. 1345

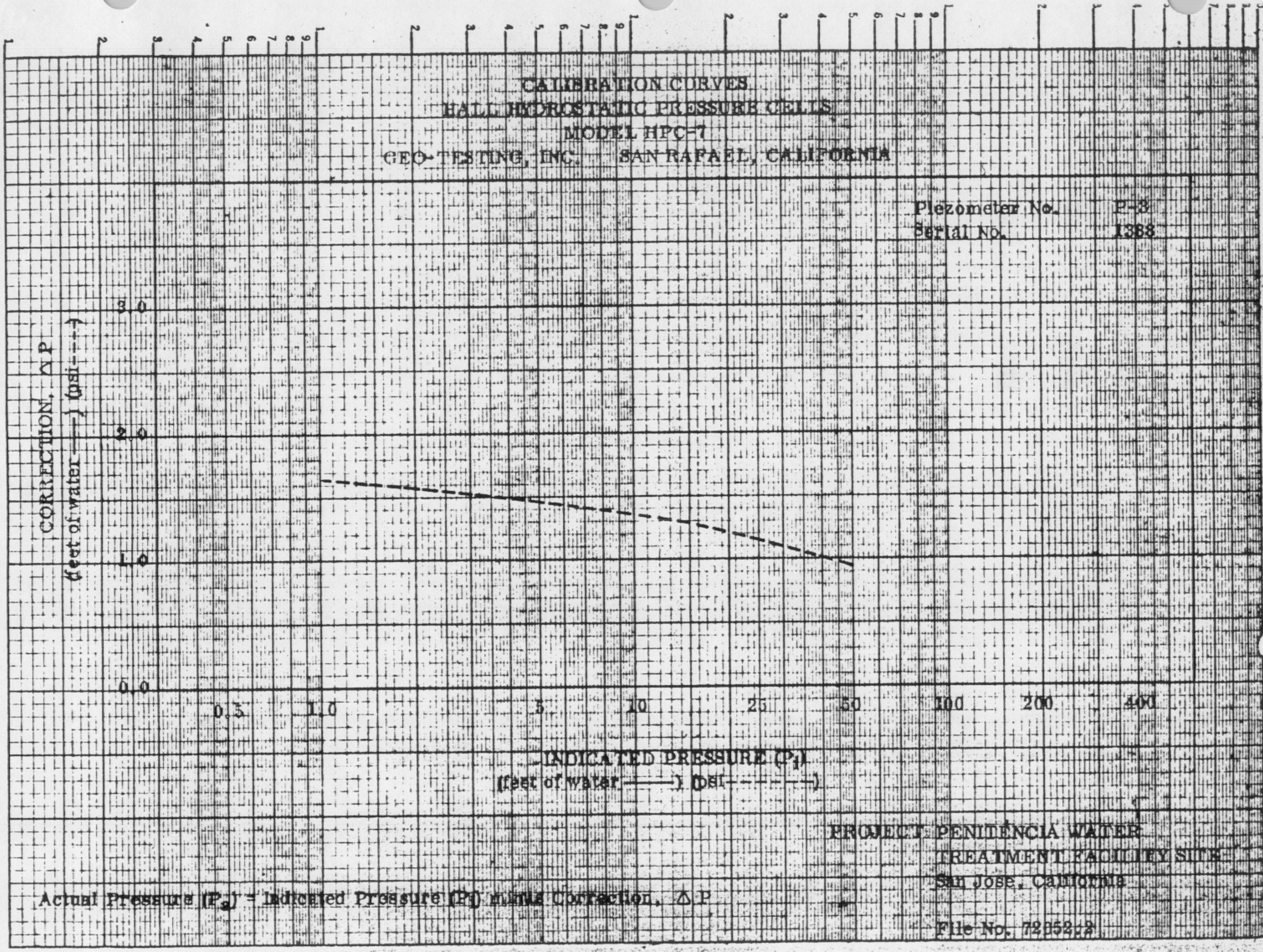


PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, Δ P

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA



Piezometer No. P-3
Serial No. 1388

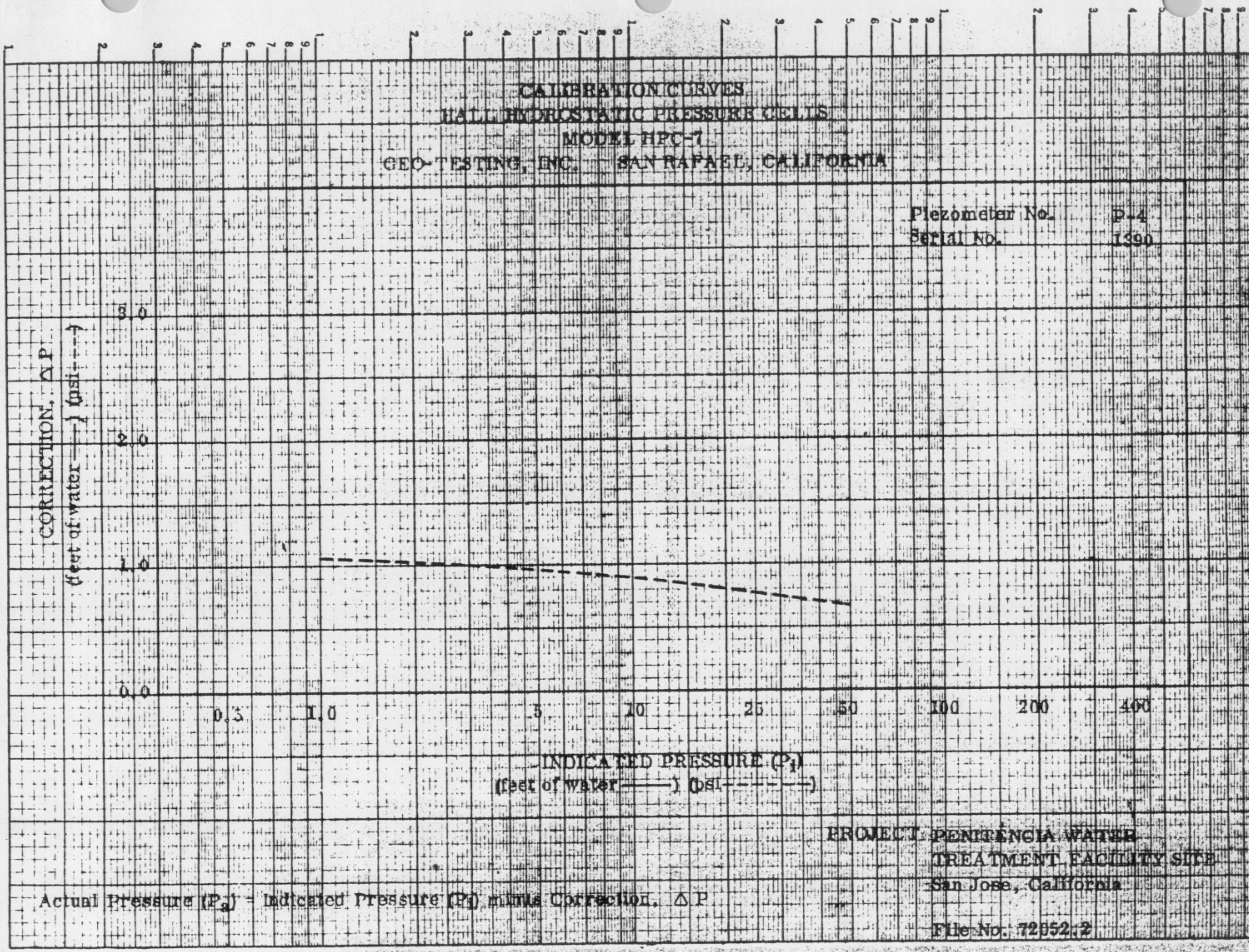
PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052-2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA

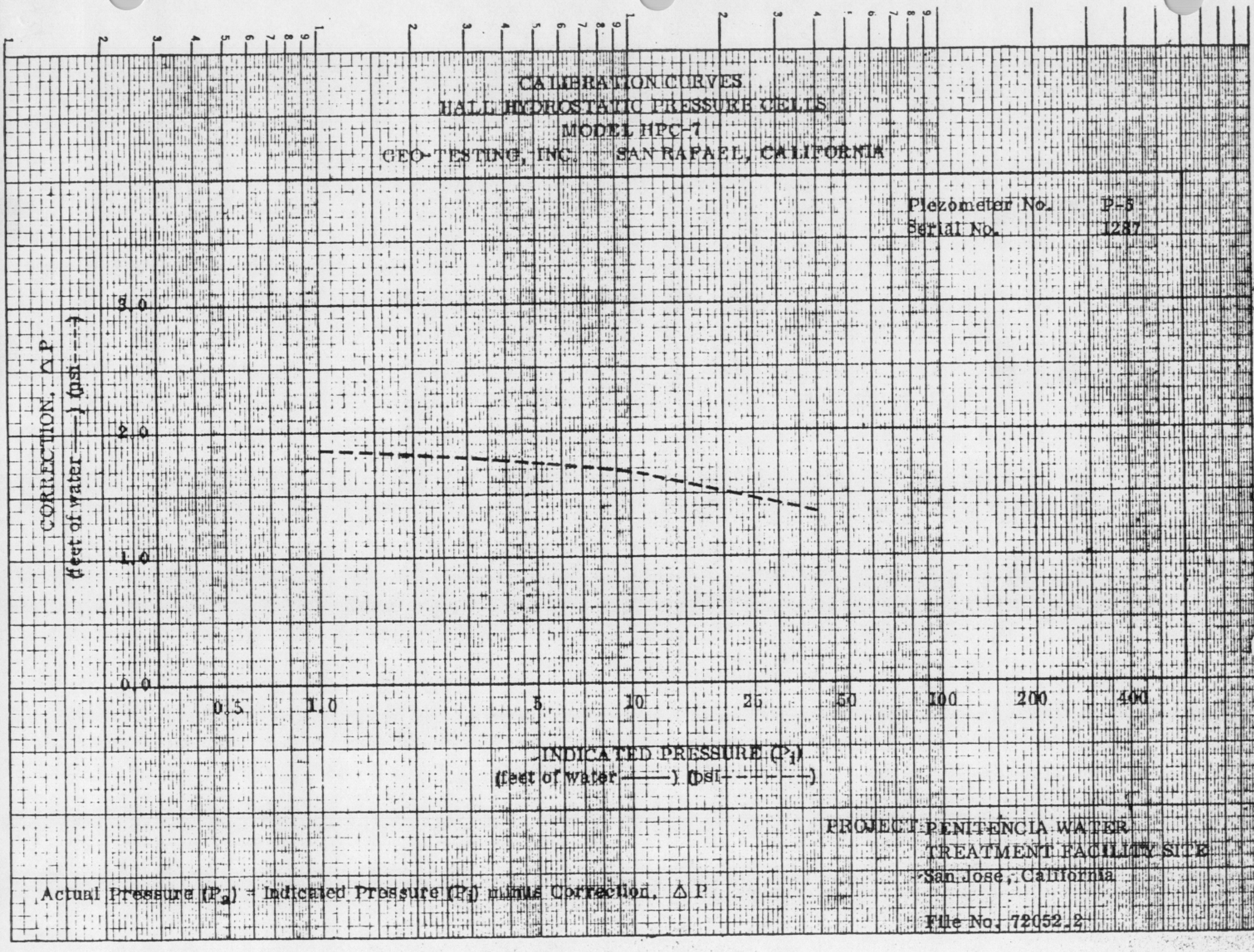
Piezometer No. P-4
Serial No. 1390



PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP



CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA

Piezometer No. P-5
Serial No. 1287

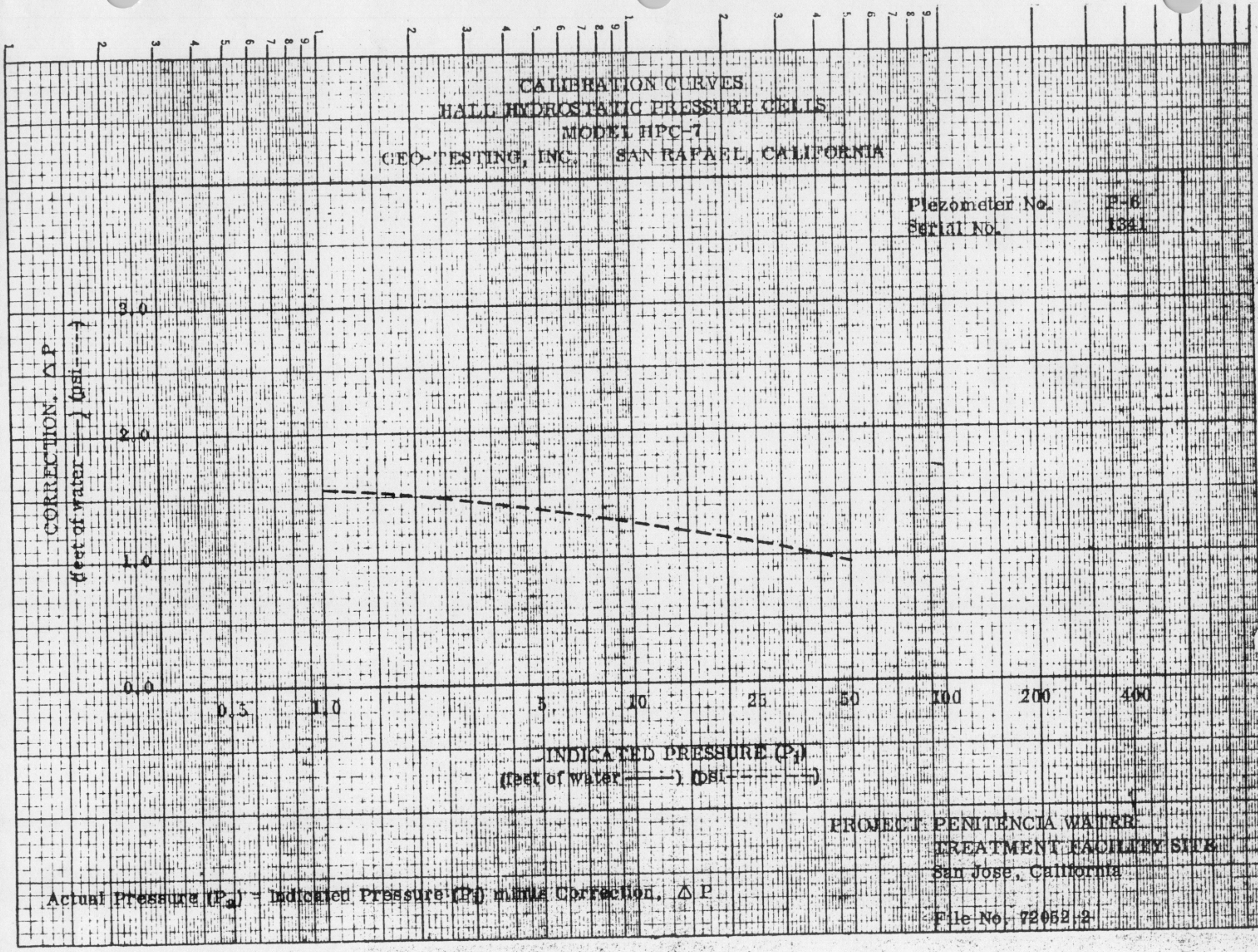
PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052-2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, Δ P

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC.   SAN RAFAEL, CALIFORNIA

Piezometer No. P-6
Serial No. 1341



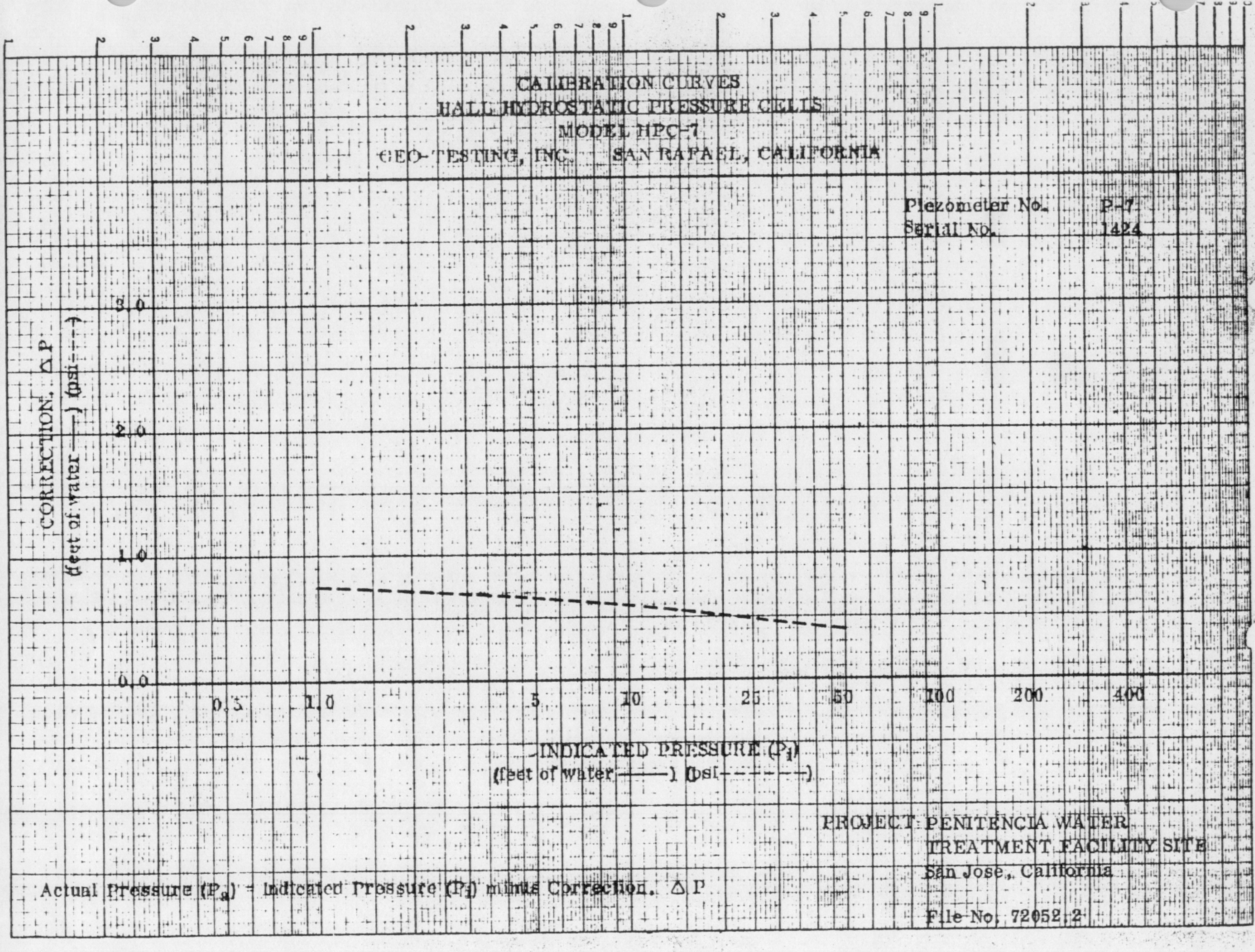
Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP

PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052-2

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC. SAN RAFAEL, CALIFORNIA

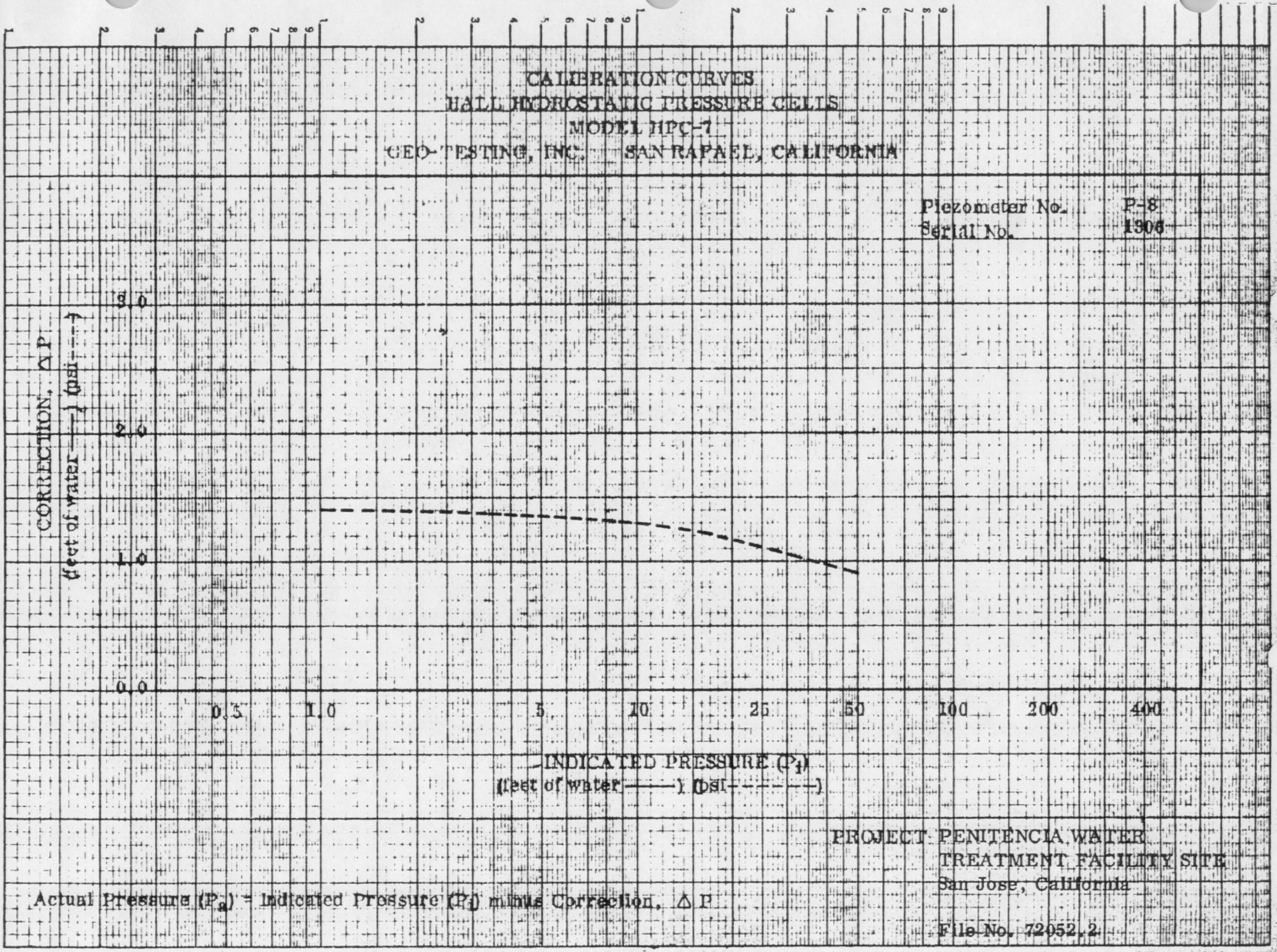
Piezometer No. P-7
Serial No. 1424



PROJECT: PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP



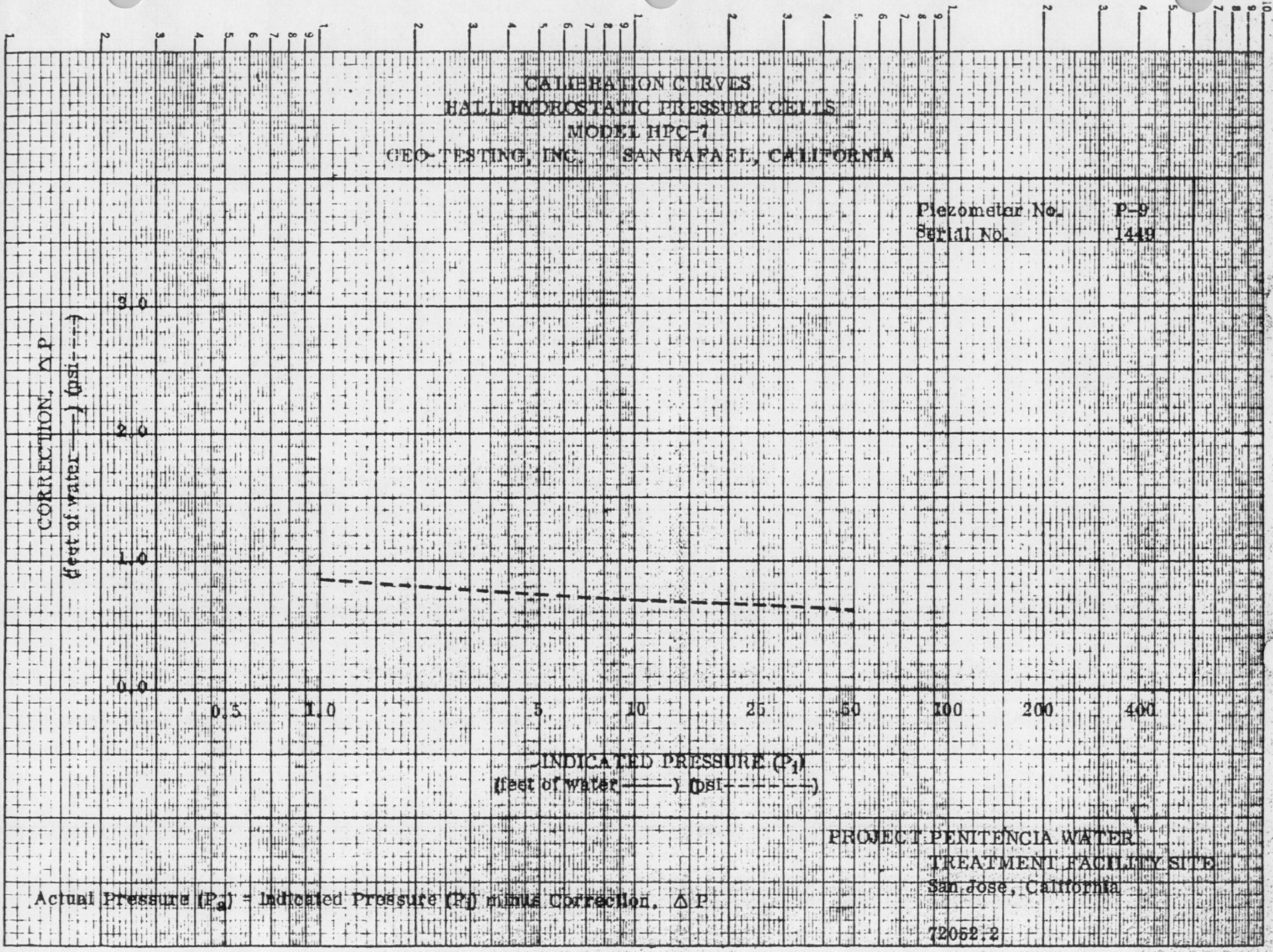
CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC. SAN RAPAEL, CALIFORNIA

Piezometer No. P-8
Serial No. 1306

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, Δ P

PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2



CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC. SAN RAFAEL, CALIFORNIA

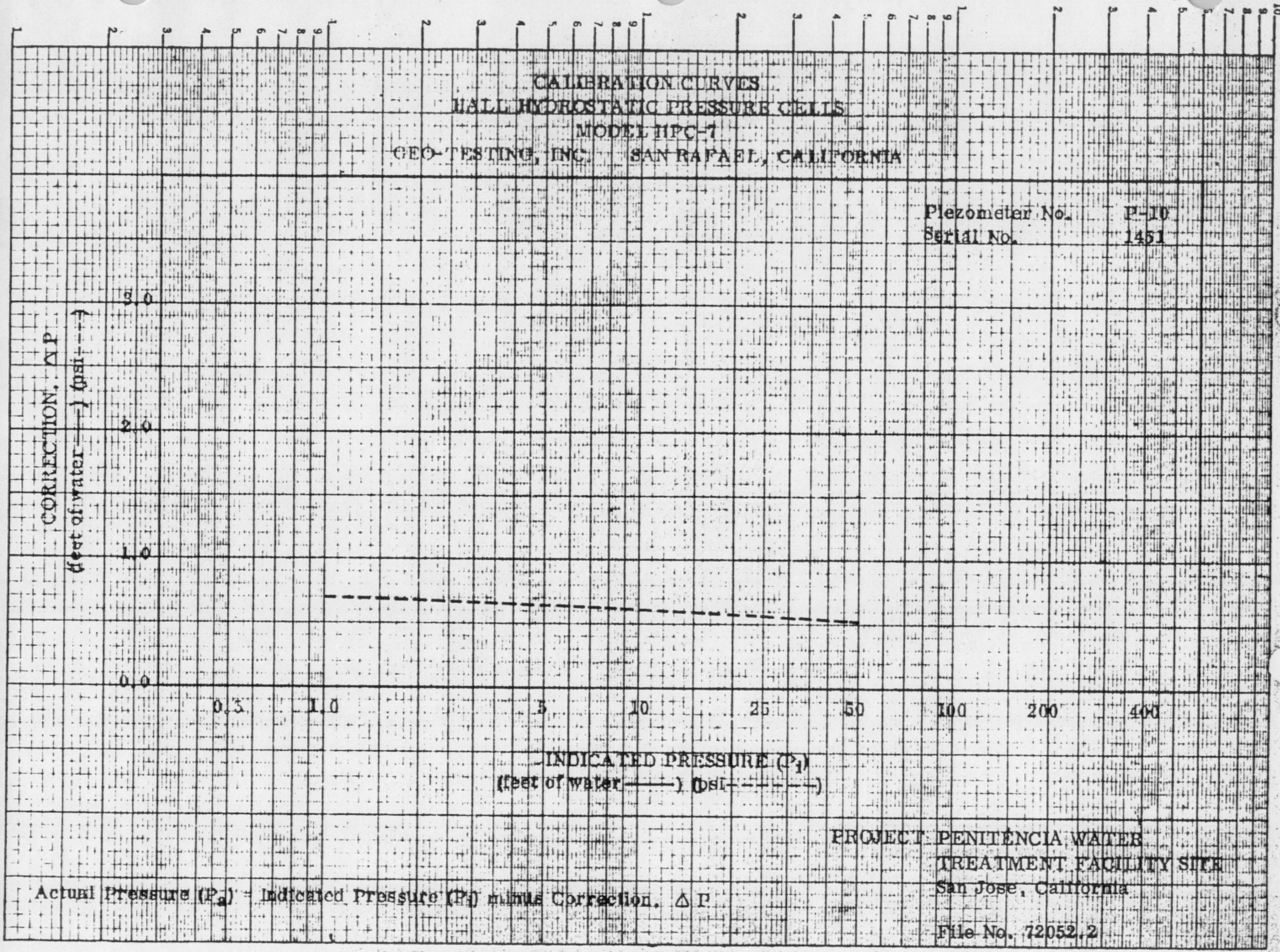
Piezometer No.	P-9
Serial No.	1449

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP

PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

72052.2

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC. SAN RAFAEL, CALIFORNIA



PROJECT PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

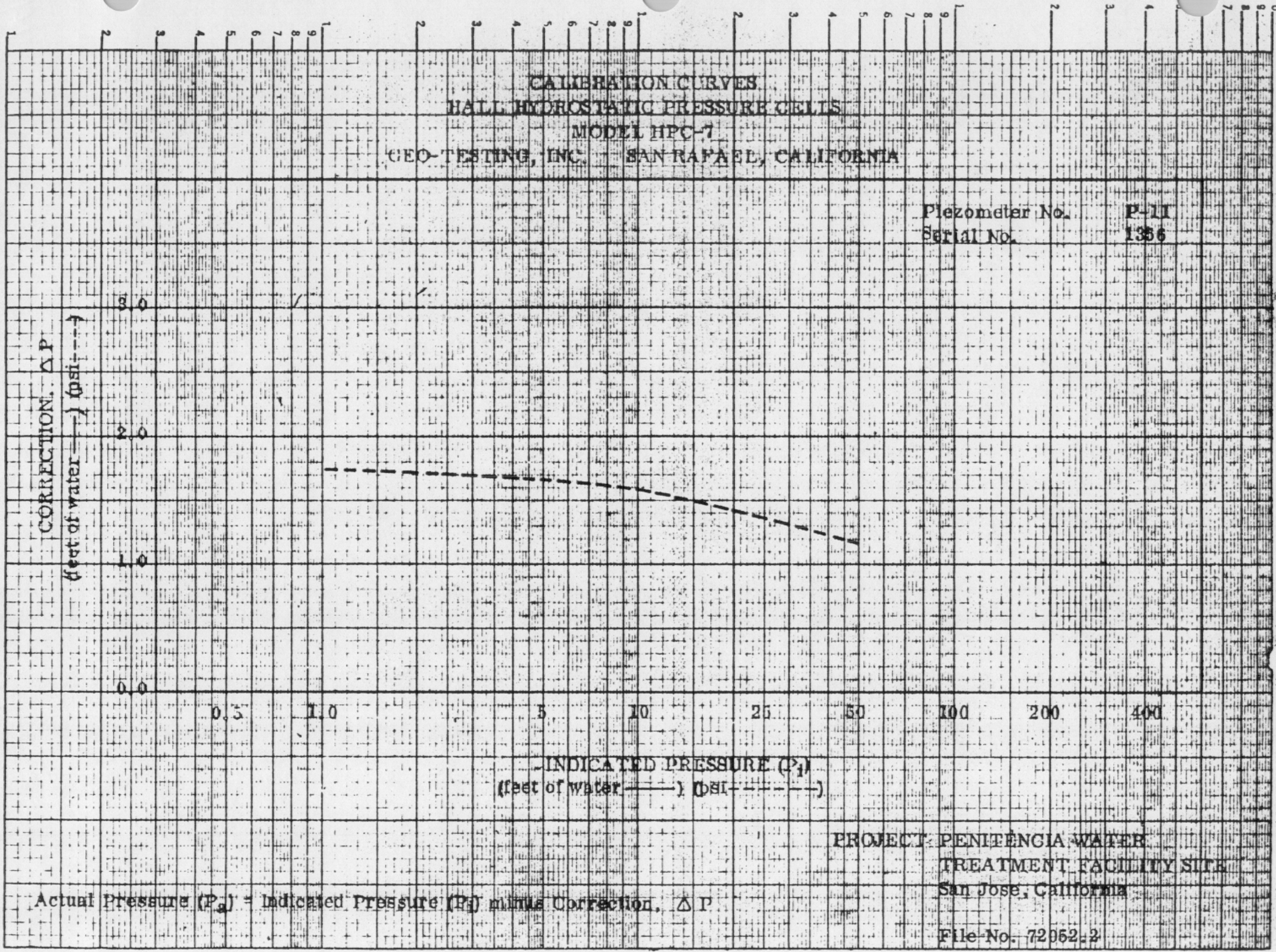
File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, ΔP

CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7

GEO-TESTING, INC. SAN RAFAEL, CALIFORNIA

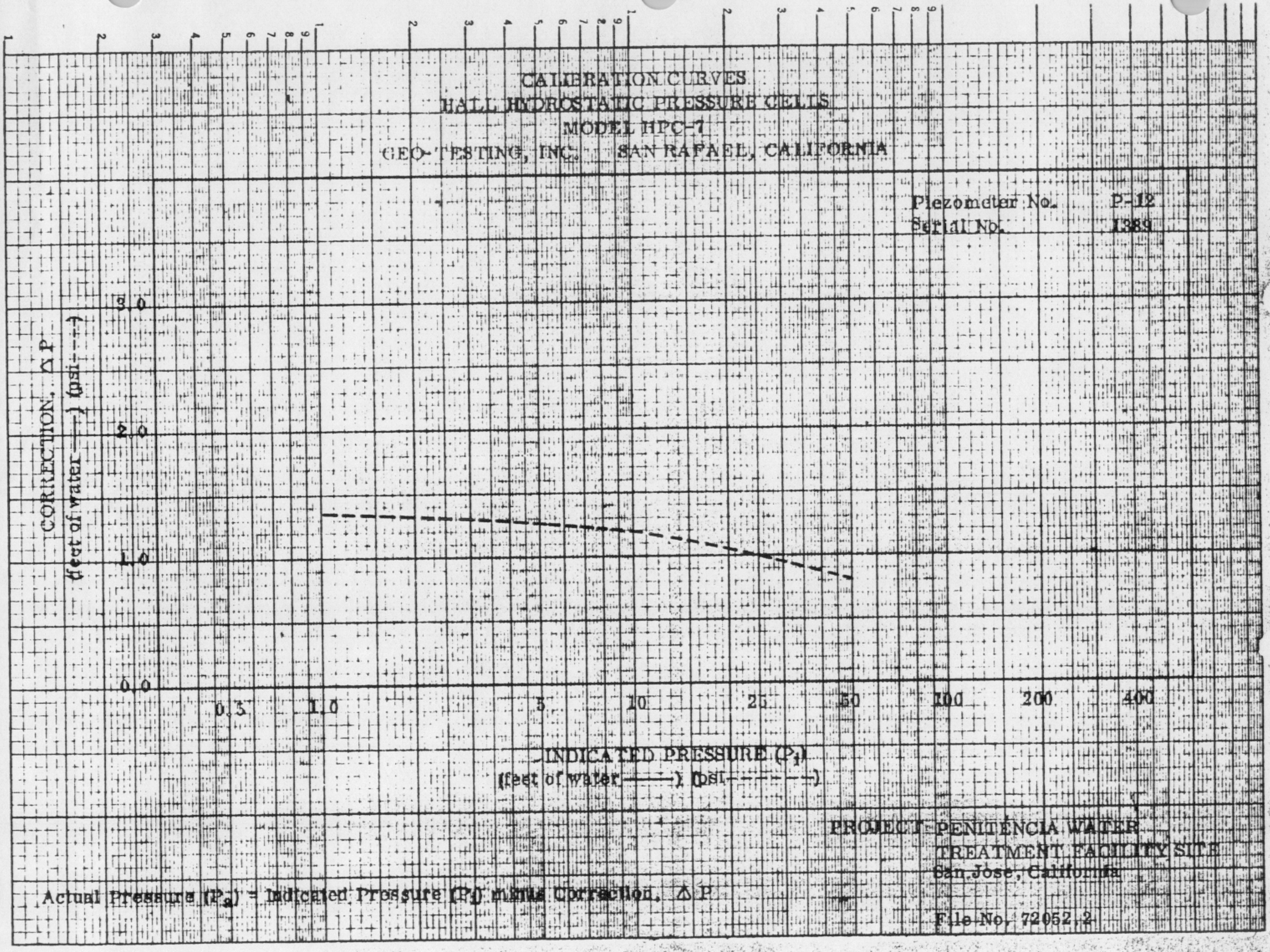
Piezometer No. P-11
Serial No. 1356



Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, △ P

PROJECT: PENITENCIA WATER
TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2



CALIBRATION CURVES
HALL HYDROSTATIC PRESSURE CELLS
MODEL HPC-7
GEO-TESTING, INC. SAN RAFAEL, CALIFORNIA

Piezometer No. P-12
Serial No. 1389

PROJECT PENITENCIA WATER TREATMENT FACILITY SITE
San Jose, California

File No. 72052.2

Actual Pressure (P_a) = Indicated Pressure (P_i) minus Correction, Δ P

HALL HYDROSTATIC PRESSURE CELL★ (Piezometer)

Purpose:

Monitor pore fluid pressure within soil-rock-water mass.

Operation:

Cells operate as closed system. Nitrogen gas is metered from the terminal console through the cell and returned to terminal. System uses minute quantities of gas sent to cell in small nylon tubes shielded by an impervious but flexible tubing.

Commonly five cells can be serviced from one multi-tube conductor. As many as 18 cells can be serviced from one multi-tube configuration.

Cell filters consist of fine but porous material. Air entry filters for pressure ranges, from 3 to 200 psi (0.2 to 14 kg/cm^2) can be furnished.

Maximum normally measurable hydrostatic pressure: 500 psi (35 kg/cm^2). Higher pressures can be measured with slightly modified console and using properly pressure-rated tubing.

Specifications:

Diameter — 1.5 in. (38 mm).

Length — 3.0 in. (75 mm).

Sensor material — stainless steel.

Case material — stainless steel or fiberglass depending on specification.

Special diameter cells made of plastic also available:

Width — 1.0 cm (0.39 in.)
Length — 2.5 cm (0.98 in.)
Thickness — 0.3 cm (0.12 in.)

Sensitivity — more than 0.01 psi (0.0007 kg/cm^2).

Accuracy — variable depending on gauge used. Normal gauge accuracy, 0.25 percent of full scale.

Gauge accuracy of 0.066 percent also readily available.

Model HPC-2 — 3.4 in. (8.5 cm) long, 4.0 in. (10.2 cm) diameter. Sensitivity — 0.005 psi (0.00035 kg/cm^2).

Model HPC-5 — entirely of stainless steel.

Model HPC-7 — outer casing of fiberglass (lower cost).

Special Features:

Leads can be placed on any gradient including reverse gradients.

Unspliced signal carrying up to 5000 ft (1,525 m).

Pneumatic system (no need for purging, no freezing or algae formation).

Pressure sensed under essentially a "no volume change" condition.

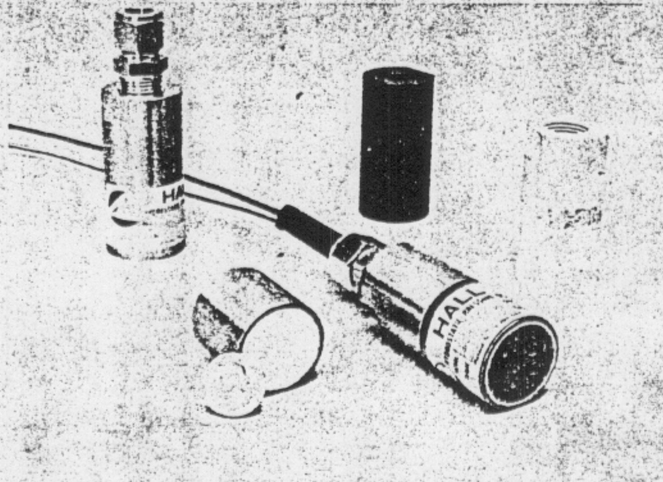
Hydrodynamic pressures (such as from atomic blasts) can be monitored with these cells if on continuous monitoring cycle. For better response time, tubing should be less than 100 m long.

Data Acquisition:

Portable or permanent readout stations can be installed.

Pressure can be observed in following ways:

- Visual reading from dial gauge.
- Recording on circular or strip charts (1-31 days).
- Digital printout (intermittent or continuous).
- Converted to radio signal transmitted to receiver for reconversion to digital display or printout.



★ Patented in the United States and Canada

PWTP PIEZO EQUATIONS

Depth to Water = (Tip Depth) - Pa .2.308

Where Pa = Pi-P

Pi = indicated pressure, psi

P = correction for piezo. calibration.

Correction factors are based on calibration curves and vary with indicated pressure. Because the variance is small within small ranges of pressure and because of the small ranges of pressure recorded to date, the correction factors are kept constant.

The correction factors are as follows:

Piezo No.	1	2	3	4	5	6
Correction Factor	1.05	1.2	1.35	1.06	1.5	1.28

Piezo No.	7	8	9	10	11	12
Correction Factor	.55	1.34	.7	.7	1.43	1.23

a difference of 0.1 in the correction factor amounts to .2 ft in the calculated depth.

R. E. Tepel 6-18-80

Santa Clara Valley Water District
5750 Almaden Expressway, San Jose 95118, Phone 265-2600

Page ________ of ________

Charge No. ________

By J. MUELLER

Date 1-17-01

Subject HALL PIEZO, CALIBRATION FACTORS

Checked by ________ Date ________

PIEZO #	TO DATE GEN L PSI RANGE	CALIBR. FOR RANGE	CONSTANT ASSIGNED BY R.E. TEPEL
1	13 - 15.5	1.05 - 1.06	1.05
2	12 - 15	1.27 - 1.20	1.20
3	8 - 12	1.38 - 1.32	1.35
4	0.6 - 1.2	1.06 - 1.05	1.06
5	15 - 22	1.57 - 1.49	1.50
6	7 - 11	1.32 - 1.26	1.28
7	15 - 17	0.55 - 0.53	0.55
8	5 - 7	1.36 - 1.33	1.34
9	7 - 10	0.72 - 0.70	0.70
10	0.1 - 3	0.69 @ 1.0 - 0.67	0.70
11	18 - 21	1.45 - 1.40	1.43
12	6 - 9	1.25 - 1.22	1.23

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-1

(2) Area Adjacent to I-1, East of Wash Water Recovery Ponds

(3) Station & Offset or Coordinates -

(4) Ground Surface Elevation Directly Above Tip ~~402~~ Feet 401.09

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7 S/N 1226

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 69.0 Ft.

(8) Tip Elevation ~~333~~ 332.09

(9) Sensing Zone Elevation Range ~~330.5~~ Ft. 329.59 to ~~334.5~~ 333.59

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-1 IN HOLE 1-PZ Arn- 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~333+2.31•RDG~~ 332.09 + 2.31(Pi - 1.05)

(21) Piezometer Equation (With Conversion of Metric Readings to Customary) Calibration correction:

Pa=~~PiP~~ Pi-P, Pa="Actual" Pressure, Pi= Indicated Pressure, P=Calibration Factor=1.05

COMMENTS

Sensing zone depth range 67.5 to 71.5 Ft. Installed with P-2 in same hole adjacent to slope indicator #1.

LAST READ 12-16-82 JM 1-17-01

BY:

R. Tepel

Date: 6/18/80

Revised 11/6/86

anta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-2

(2) Area Adjacent to I-1, East of Wash Water Recovery Ponds

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~402~~ 401.09

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 40 Ft.

(8) Tip Elevation ~~362~~ Feet 361.09

(9) Sensing Zone Elevation Range ~~361~~ Ft. 361.59 to ~~362.5~~ Ft. 360.09

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-2 N HOLE 1-PZ. AM - 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~362-2.31•RDG~~ 361.09 + 2.31(P$_i$ - 1.2)

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pi=Indicated Pressure; Pa="Actual" Pressure; P=Calibration Factor=1.2

COMMENTS

Sensing zone depth 39.5-41.0 Ft.

Installed in same hole as P-1.

LAST READ 3-11-97 JM 1-17-01

BY:

R. Tepel

Date: 6/18/80
Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-3

(2) Area South of Whitman ~~Avenue~~ WAY and West of Vista Del Mar

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~351~~ Ft. 349.50

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 70 Ft.

(8) Tip Elevation ~~281~~ Ft. 279.50

(9) Sensing Zone Elevation Range ~~279~~ Ft. 277.50 to ~~282.5~~ Ft. 281.00

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-3 IN HOLE 2-PZ. AM- 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~281+2.31•RDG~~ $279.50 + 2.31(P_i - 1.35)$

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa="Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.35

COMMENTS

Installed adjacent to Inclinometer I-2 in same hole with P-4.

Sensing zone depth 68.5 to 72.0 Ft.

JM 1-17-01

BY:

R. Tepel

Date: 6-18-80
Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-4

(2) Area South of Whitman ~~Avenue~~ WAY and West of Vista Del Mar

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~351~~ Ft. 349.50

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 40 Ft.

(8) Tip Elevation ~~311~~ Ft. 309.50

(9) Sensing Zone Elevation Range ~~309~~ Ft. 307.50 to ~~313~~ Ft. 311.50

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-4 IN HOLE I-2 AML 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~311+2.31•RDG~~ 309.50+ 2.31(Pi - 1.06)

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa= "Actual" Pressure, Pi=Indicated Pressure; P=Calibration Factor = 1.06

COMMENTS

Installed adjacent to I-2 in same hole with P-3

Sensing zone depth 38.0 to 42.0 Ft.

JM 1-29-01

BY: R. Tepel

Date: 6-18-80
Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-5

(2) Area West of SBA Tank and North of Control Building

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~453~~ Ft. 452.14

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 70 Ft.

(8) Tip Elevation ~~383~~ Ft. 382.14

(9) Sensing Zone Elevation Range ~~381~~ Ft. 380.14 to ~~384.5~~ Ft. 383.64

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-5 IN HOLE 3-PZ. AM 4|22|96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE= ~~383+2.31•RDG~~ 382.14 + 2.31 $(P_i - 1.5)$

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa= "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.5

COMMENTS

Installed adjacent to I-3 in same hole with P-6

Sensing zone depth 68.5 to 72.0 Ft.

JM 1-29-01

BY: R. Tepel

Date: 6-18-80
Revised 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-6

(2) Area West of SBA Tank and North of Control Building

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~455~~ Ft. 452.14

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 38.5 Ft.

(8) Tip Elevation ~~414.5~~ Ft. 413.64

(9) Sensing Zone Elevation Range ~~414~~ Ft. 412.6 to ~~416~~ Ft. 415.1

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-6 IN HOLE 3-PZ AMH 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~414.5 + 2.31•RDG~~ 413.6 + 2.31(Pi - 1.28)

(21) ~~Piezometer Equation (With Conversion of Metric Reading to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa="Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.28

COMMENTS

Installed in same hole with P-5 adjacent to I-3

Sensing zone depth range 37.0 to ~~39.0~~ 39.5 Ft.

JM 1-29-01

BY:

R. Tepel

Date: 6-18-80

Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-7

(2) Area Northeasterly of clear well

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~442~~ Ft. 442.56

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 70 Ft.

(8) Tip Elevation ~~372~~ Ft. 372.56

(9) Sensing Zone Elevation Range ~~370~~ Ft. 370.6 to ~~373.5~~ Ft. 374.1

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-7 IN HOLE 4-PZ. AM÷ 4221 9io

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~372 + 2.31•RDG~~ 372.6 + 2.31 (Pi - 0.55)

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa= "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 0.55

COMMENTS

Installed adjacent to I-4 in same hole as P-8

Sensing zone depth range 68.5 to 72.0 Ft.

JM 1-29-01

BY: R. Tepel

Date: 6-18-80

Revised 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-8

(2) Area Northeasterly of clear well

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~442~~ Ft. 442.56

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 39 Ft.

(8) Tip Elevation ~~403~~ Ft. 403.56

(9) Sensing Zone Elevation Range ~~∂370~~ Ft. 402.6 ~~-22~~ to ~~∂373.5~~ Ft. 404.1 ~~403.5~~

BASED ON EBH LETTER OF 11/14/72
AMH
4/22/96

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-8 IN HOLE 4-PZ AMH 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~403 + 2.31 RDG~~ 403.6 + 2.31 (Pi - 1.34)

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa = "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.34

COMMENTS

Sensing zone depth ~~68.5 to 72.0 Ft. ?~~ 38.5-40.0 FT BASED ON EBH LETTER

Installed adjacent to I-4 in same hole as P-7 OF 11/14/72 AMH 4/22/96

JM 1-29-01

BY:

R. Tepel

Date: 6-18-80
Revised 11-6-86

ta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-9

(2) Area Northerly of Whitman ~~Avenue~~ WAY, EAST OF BAY LAUREL LANE AML 4/22/96

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~345~~ Ft. 345.30

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 70 Ft.

(8) Tip Elevation ~~275~~ Ft. 275.30

(9) Sensing Zone Elevation Range ~~273~~ Ft. 273.3 to ~~276.5~~ Ft. 276.8

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-9 IN HOLE 5-PZ AML 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~275 + 2.31•RDG~~ 275.3 + 2.31 $(P_i - 0.70)$

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa = "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 0.7

COMMENTS

Sensing zone depth range 68.5 to 72.0 Ft.

Installed adjacent to I-5 in same hole as P-10

JM 1-29-01

BY: R. Tepel

Date: 6-18-80
Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-10

(2) Area North of Whitman ~~Avenue~~ WAY, EAST OF BAY LAUREL LANE AMH 4/22/96

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~345~~ Ft. 345.30

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 40 Ft.

(8) Tip Elevation ~~305~~ Ft. 305.30

(9) Sensing Zone Elevation Range ~~304~~ Ft. 304.3 to ~~306.5~~ Ft. 306.8

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-10 IN HOLE 5-PZ. AMH 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~305 + 2.31•RDG~~ 305.3 + 2.31 (Pi - 0.70)

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary~~) Calibration Correction:

Pa=Pi-P; Pa = "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 0.7

COMMENTS

Sensing zone depth range 38.5 to 41.0 Ft.

Installed adjacent to I-5 in same hole as P-9

JM 1-29-01

BY: R. Tepel

Date: 6-18-80
Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-11

(2) Area East of Bay Laurel Lane, WEST OF SLUDGE DRYING BEDS. AMH 4/22/96

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~359~~ Ft. 358.99

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 70 Ft.

(8) Tip Elevation ~~289~~ Ft. 388.99

(9) Sensing Zone Elevation Range 287 Ft. ✓ to 290.5 Ft. ✓

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-11 IN HOLE 6-PZ AMH 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~289 + 2.31• RDG~~ 289.0 + 2.31$(P_i - 1.43)$

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa="Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.43

COMMENTS

Sensing zone depth range 68.5 to 72.0 Ft

Installed adjacent to I-6 in same hole as P-12

JM 1-29-01

BY:

R. Tepel

Date: 6-18-80

Revised 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-12

(2) Area East of Bay Laurel Lane, WEST OF SLUDGE DRYING BEDS. AMH 4/22/96

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~359~~ Ft. 358.99

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model Earl B. Hall, HPC-7

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 40 Ft.

(8) Tip Elevation ~~319~~ Ft. 318.99

(9) Sensing Zone Elevation Range 318 Ft. ✓ to 320.5 Ft. ✓

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed Sept. 1972 By Geo-Testing, Inc.

(16) Report Describing Installation Letter 14 November 1972 to Watkins and Iwamura

(17) Identification When Installed P-12 IN HOLE 6-PZ. AMH 4/22/96

(18) Purpose

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly

(20) Piezometer Equation (Customary Units) PSE = ~~319 + 2.31•RDG~~ $319.0 + 2.31(P_i - 1.23)$

(21) ~~Piezometer Equation (With Conversion of Metric Readings to Customary)~~ Calibration Correction:

Pa=Pi-P; Pa = "Actual" Pressure; Pi=Indicated Pressure; P=Calibration Factor = 1.23

COMMENTS

Sensing zone depth range 38.5 to 41.0 Ft.

Installed adjacent to I-6 in same hole as P-11

JM 1-29-01

BY: R. Tepel

Date: 6-18-80

Revised 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-13

(2) Area Near Piccolo Residence; Whitman Way

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~284~~ Ft. 284.46

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO No. 514177; S/N 26573

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 45 Ft.

(8) Tip Elevation ~~239~~ Ft. 239.46

(9) Sensing Zone Elevation Range ~~239~~ Ft. 239.5 to ~~245~~ Ft. 245.5

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/19/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-13 in hole 7PZ

(18) Purpose Monitor pore pressure near slide surface at toe

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~239 + 2.31•RDG~~ 239.5 + 2.31 × Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE [illegible]

ADJACENT TO I-10

AMH 4/22/96

JM 1-29-01

BY:

R. Kimmel & R. Tepel Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-14

(2) Area Southwest Corner of Whitman Way and Vista Del Mar

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~375~~ Ft. 375.66

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26574

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 125 Ft.

(8) Tip Elevation ~~250~~ Ft. 250.66

(9) Sensing Zone Elevation Range ~~249~~ Ft. 249.7 to ~~252~~ Ft. 252.7

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/8/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-14 in Hole No. 8-PZ

(18) Purpose Monitor pore pressure near slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = 250.7 + 2.31 x (Reading)

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE [illegible] 123-126 [illegible]

[illegible] P-14 INSTALLED [illegible]

[illegible]

JM 1-29-01

BY:

R. Kimmel and R. Tepel

Date: 11-6-86

ta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-15

(2) Area Southwest Corner of Whitman Way and Vista Del Mar

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~375~~ Ft. 375.66

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26575

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 90 Ft.

(8) Tip Elevation ~~285~~ Ft. 285.7

(9) Sensing Zone Elevation Range 285.7Ft. to 290.7Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/11/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-15 in Hole No. 8-PZ

(18) Purpose Monitor pore pressure above slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = 285.7+ 2.31 RDG

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

[illegible]

[illegible] SAME [illegible]

[illegible]

1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-16
(2) Area North End of Sludge Drying Beds
(3) Station & Offset or Coordinates
(4) Ground Surface Elevation Directly Above Tip ~~389~~ Ft. 390.13
(5) Type ☒ Pneumatic ☐ Open Well
(6) Manufacturer and Model SINCO #514177 S/N 26576

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 144 Ft.
(8) Tip Elevation ~~245~~ Ft. 246.13
(9) Sensing Zone Elevation Range ~~265~~ Ft. 246.1 to ~~251~~ Ft. 252.1

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point
(11) Reference Point Elevation
(12) Tip Depth Below R.P. Below Ground Surface
(13) Tip Elevation
(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/20/86 By Pitcher Drilling Company
(16) Report Describing Installation None
(17) Identification When Installed P-16 in Hole #9-PZ
(18) Purpose Monitor pre pressure near slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly; January, April, July, October
(20) Piezometer Equation (Customary Units) PSE = ~~245 + 2.31•RDG~~ 246.1 + 2.31 × RDG
(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE 138-144 FT.
ADJACENT TO I-3. INSTALLED IN SAME HOLE AS P-17
AND [illegible].
DM 1-29-01

BY:
R. Kimmel and R. Tepel
Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-17

(2) Area North End of Sludge Drying Beds

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~389~~ Ft. 390.13

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26577

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 80 Ft.

(8) Tip Elevation ~~309~~ Ft. 310.13

(9) Sensing Zone Elevation Range ~~307~~ Ft. 308.1 to ~~314~~ Ft. 315.1

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/21/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-17 in Hole #9-PZ

(18) Purpose Monitor pore pressure above slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~309~~ 310.1 + 2.31xRDG

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE 75.0-82.0 FT.
ADJUSTMENT TO EL'S. INCLUDED IN [illegible]
[illegible]

JM 1-29-01

BY:

R. Kimmel & R. Tepel

Date: 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-18

(2) Area 250 Feet Northeast of clear well

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~443~~ Feet 443.47

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26578

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 195.5 Feet

(8) Tip Elevation ~~247.5~~ Feet 247.97

(9) Sensing Zone Elevation Range 248.0 ~~247.5~~ Feet to ~~251~~ Feet 251.6

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/22/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-18 in Hole #10-PZ

(18) Purpose Monitor pore pressure near slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~247.5~~ 248.0 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

[illegible]

[illegible] P-4. Installed in same hole as P-19.

[illegible]

JM 1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-19

(2) Area 250 Feet ~~North~~ EAST of Clear Well AMH 4/22/96

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~443~~ Feet 443.47

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26579

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 145 Feet

(8) Tip Elevation ~~298~~ Feet 298.47

(9) Sensing Zone Elevation Range ~~295~~ Feet 295.5 to ~~306~~ 306.5 Feet *(See Below)

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/22/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-19 in Hole #10-PZ

(18) Purpose Monitor pore pressure above slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~298~~ 298.5 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

*Top of sand not known - top of sand + bentonite pellets is

313 Feet - Estimate 3 Feet of Pellets

[illegible] 137.0-148.[illegible]

[illegible] 18. [illegible]

BY: JM 1-29-01

R. Kimmel and R. Tepel Date: 11-6-86

Santa Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-20

(2) Area Hill Near San Jose Water Company Dutard Reservoir

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~449~~ Feet 449.26

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26580

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 115 Feet

(8) Tip Elevation ~~334~~ Feet 334.26

(9) Sensing Zone Elevation Range ~~332.5~~ 332.8 Ft. to ~~337~~ 337.3 Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/6/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-20 in Hole #11-PZ

(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly, January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~334~~ 334.3 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE 112.0-116.5 FT.

ADJACENT TO I-17. INSTALLED IN SAME HOLE AS P-21.

AMH 1/22/96

JM 1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-21

(2) Area Hill Near San Jose Water Company Dutard Reservoir

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~449~~ Ft. 449.26

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26581

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 87 Ft.

(8) Tip Elevation ~~362~~ Ft. 362.26

(9) Sensing Zone Elevation Range ~~357~~ Ft. 357.3 to ~~366~~ Ft. 366.3

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/6/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-21 in Hole #11-PZ

(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~362~~ 362.3 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE 83.0-92.0 FT

EQUIVALENT TO I-7. INSTALLED IN SAME HOLE AS P-2[illegible]

AND [illegible]

JM 1-29-01

BY:

R. Kimmel and R. Tepel

Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET
FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-22
(2) Area Near San Jose Water Company Redwood Tank (Above Dutard Reservoir)
(3) Station & Offset or Coordinates
(4) Ground Surface Elevation Directly Above Tip ~~536~~ Ft. 535.23
(5) Type ☒ Pneumatic ☐ Open Well
(6) Manufacturer and Model SINCO #514177 S/N 26582

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 230 Ft.
(8) Tip Elevation ~~306~~ Ft. 305.23
(9) Sensing Zone Elevation Range ~~304.5~~ 303.7 Ft. to ~~310~~ 309.2 Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point
(11) Reference Point Elevation
(12) Tip Depth Below R.P. Below Ground Surface
(13) Tip Elevation
(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 7/30/86 By Pitcher Drilling Company
(16) Report Describing Installation None
(17) Identification When Installed P-22 in Hole #12-PZ
(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October
(20) Piezometer Equation (Customary Units) PSE = ~~306~~ 305.2 + 2.31 x Reading
(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE [illegible] RANGE [illegible]
ADJACENT TO I-[illegible] INSTALLED IN SAME HOLE AS P-22
[illegible] 4/22/96

JM 1-29-01

BY:

R. Kimmel and R. Tepel

Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-23

(2) Area Near San Jose Water Company Redwood Tank (Above Dutard Reservoir)

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~536~~ Ft. 535.23

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26583

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 135 Ft.

(8) Tip Elevation ~~401~~ Ft. 400.23

(9) Sensing Zone Elevation Range ~~395~~ 394.2 Ft. to ~~404~~ 403.2 Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 7/30/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-23 in Hole #12-PZ

(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~401~~ 400.2 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENS[illegible]

[illegible] INSTALLED IN SAME HOLE AS P-22

[illegible]

JM 1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-24

(2) Area End of Telegraph Drive between Sidewalk and Curb

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~414~~ Ft. 414.64

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26584

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 145 Ft.

(8) Tip Elevation ~~269~~ Ft. 269.64

(9) Sensing Zone Elevation Range ~~266~~ Ft. 266.6 to ~~272~~ Ft. 272.6

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/13/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-24 in Hole #13-PZ

(18) Purpose Monitor pore pressure near slide surface.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~269~~ 269.6 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE 142.0 - 148.0 --

ADJACENT TO I-15.

AMH 4/22/91.

JM 1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

anta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-25

(2) Area Off Suncrest Avenue on McCredes Parcel Above Plant

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~552~~ Ft. 549.85

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26831

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 230 Ft.

(8) Tip Elevation ~~322~~ Ft. 319.85

(9) Sensing Zone Elevation Range ~~321~~ 318.9 Ft. to ~~326~~ 323.9 Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/27/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-25 in Hole #14-PZ

(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~322~~ 319.9 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENSING ZONE DEPTH RANGE = 226.0 - 231.0 FT

ADJACENT TO I-10. INSTALLED IN SAME HOLE AS P-26

[illegible] I-12 [illegible]

LAST READ 9-22-92 D/T NEW LOCK ON GATE JM 1-29-01

BY:

R. Kimmel and R. Tepel Date: 11-6-86

nta Clara Valley Water District

PIEZOMETER DATA SHEET

FC 613 (12-27-79)

LOCATION AND TYPE

(1) Facility PENITENCIA WATER TREATMENT PLANT Piezometer No. P-26

(2) Area Off Suncrest Avenue on McCredes Parcel Above Plant

(3) Station & Offset or Coordinates

(4) Ground Surface Elevation Directly Above Tip ~~552~~ Ft. 549.85

(5) Type ☒ Pneumatic ☐ Open Well

(6) Manufacturer and Model SINCO #514177 S/N 26830

REFERENCE DATA FOR PNEUMATIC PIEZOMETERS

(7) Tip Depth Below Ground Surface 134 Ft.

(8) Tip Elevation ~~418~~ Ft. 415.85

(9) Sensing Zone Elevation Range ~~418~~ 415.9 Ft. to ~~421.5~~ 419.4 Ft.

REFERENCE DATA FOR OPEN WELL PIEZOMETERS

(10) Reference Point

(11) Reference Point Elevation

(12) Tip Depth Below R.P. Below Ground Surface

(13) Tip Elevation

(14) Sensing Zone Elevation Range to

INSTALLATION

(15) Date Installed 8/28/86 By Pitcher Drilling Company

(16) Report Describing Installation None

(17) Identification When Installed P-26 in Hole #14-PZ

(18) Purpose Monitor pore pressure near head of slide.

READING AND CALCULATION DATA

(19) Scheduled Readings Quarterly: January, April, July, October

(20) Piezometer Equation (Customary Units) PSE = ~~418~~ 415.9 + 2.31 x Reading

(21) Piezometer Equation (With Conversion of Metric Readings to Customary)

COMMENTS

SENS [illegible] 130.5 - 134.0 ft.

[illegible] SAME [illegible] P-25

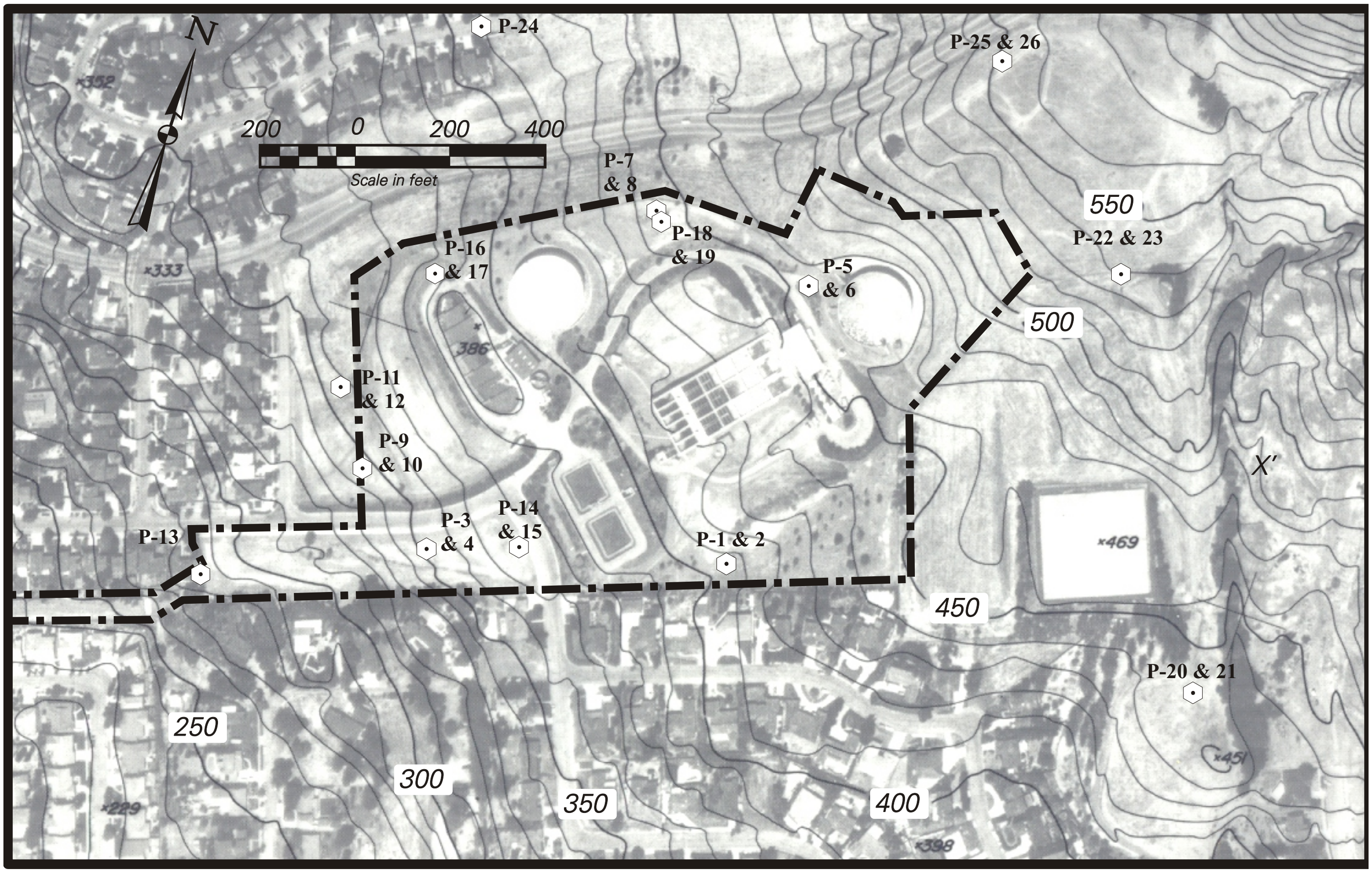
[illegible]

LAST READ 9-22-92 DUE TO NEW LOCK ON GATE JM 1-29-01

BY:

R. Kimmel and R. Tepel

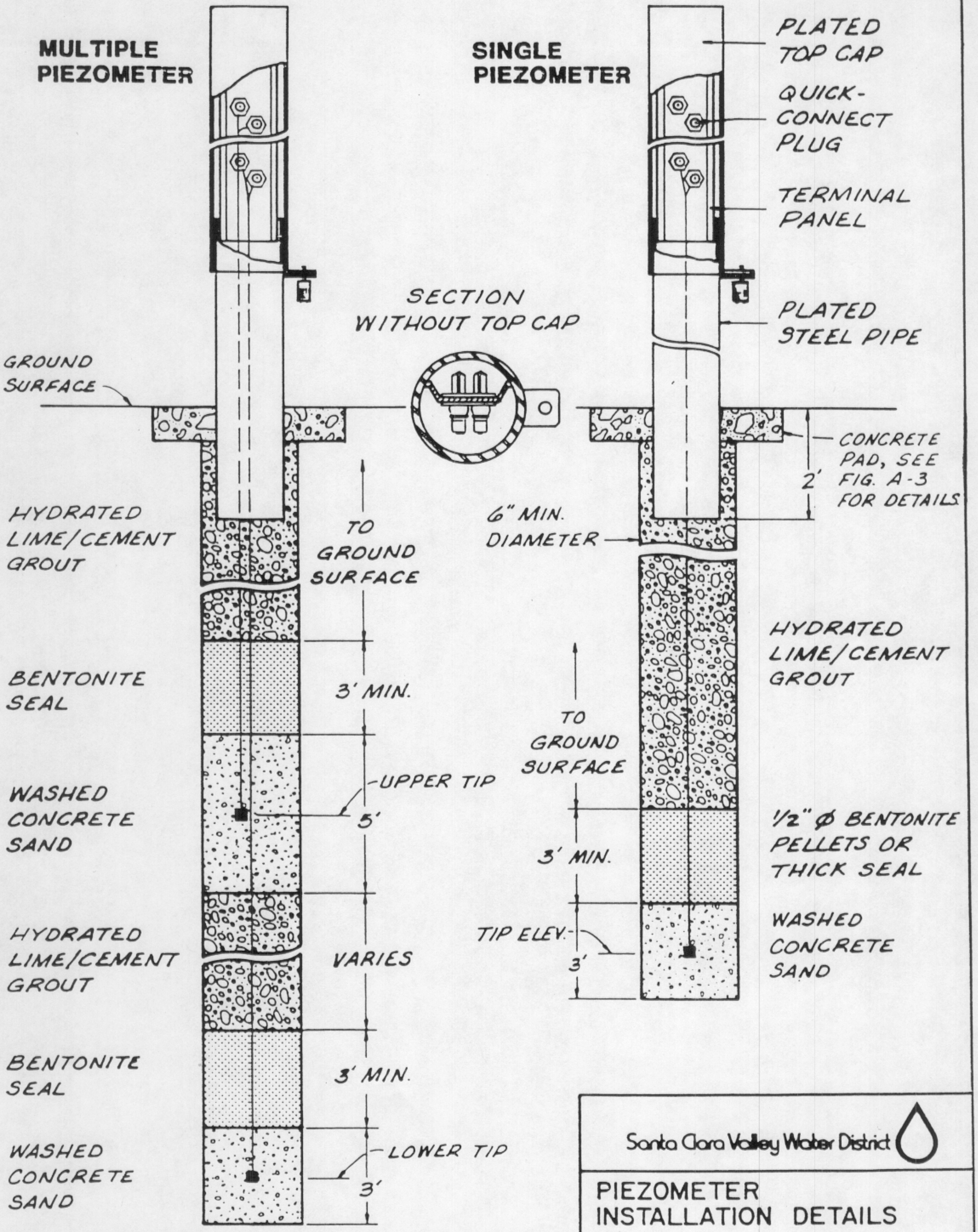
Date: 11-6-86



Santa Clara Valley Water District

Penitencia Creek Landslide Piezometer Locations

Figure B-1

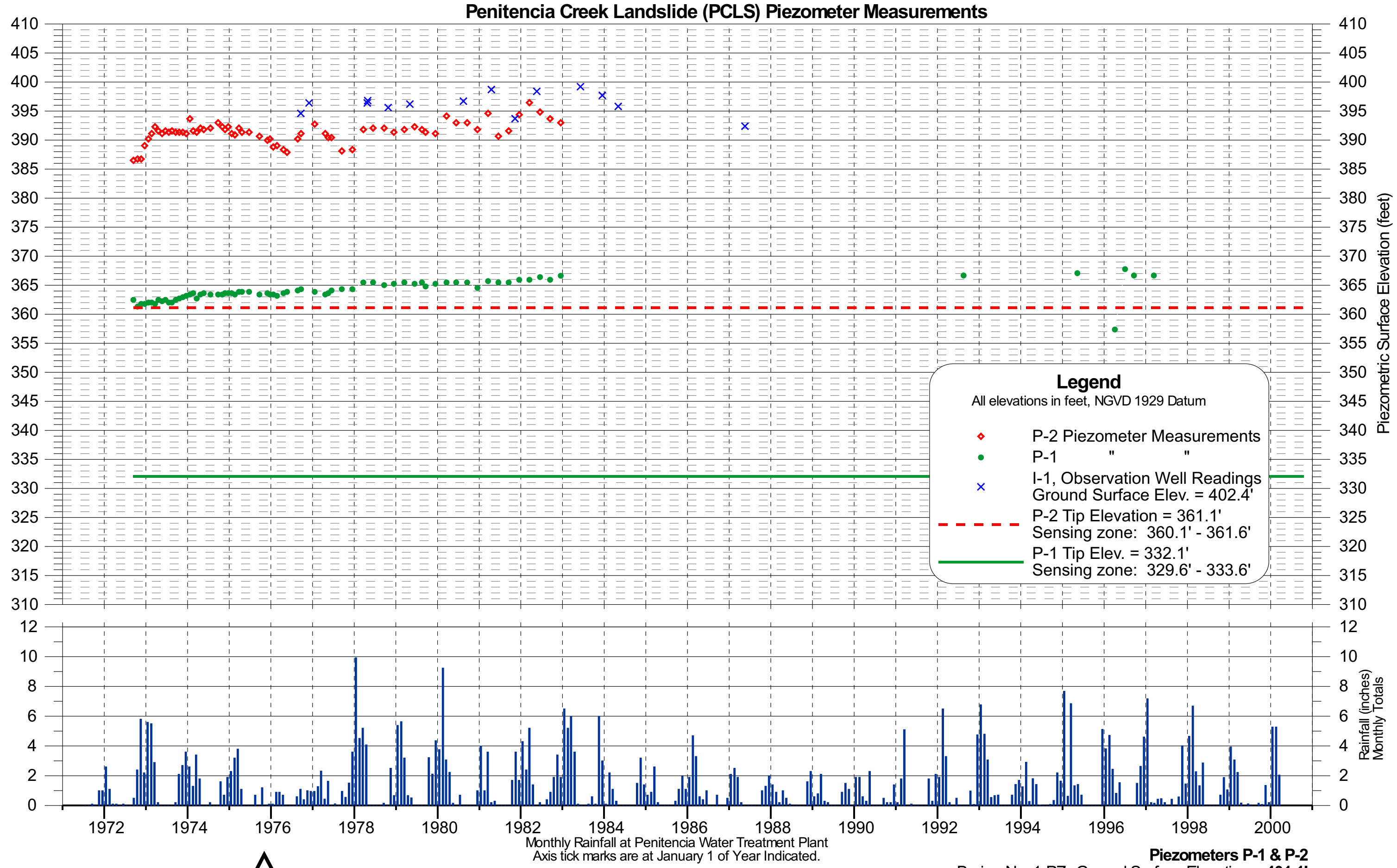


NOTE: SEALING OF WELL TO CONFORM TO S.C.V.W.D. REQUIREMENTS.

Santa Clara Valley Water District

PIEZOMETER
INSTALLATION DETAILS

FIGURE B-2



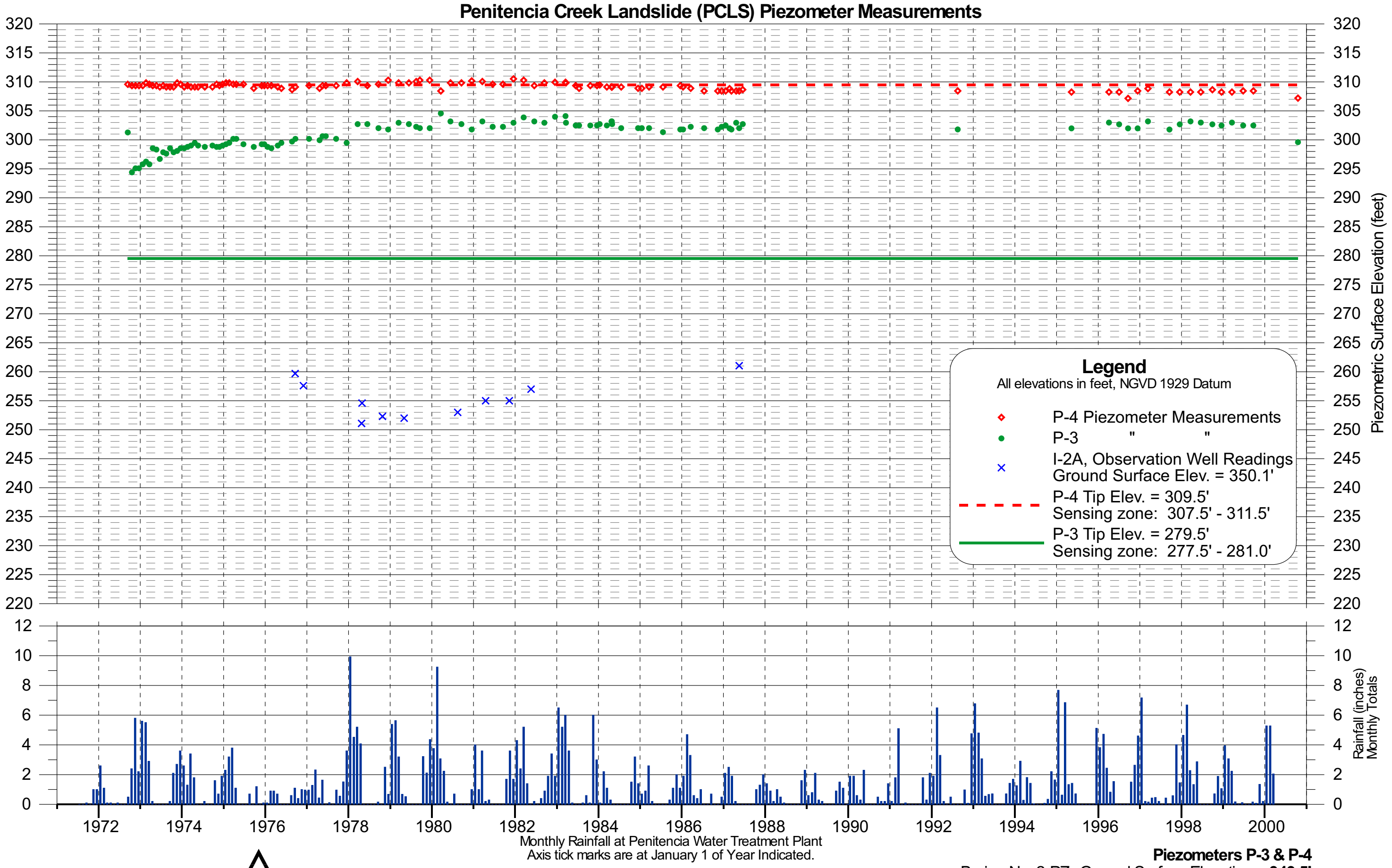
Penitencia Creek Landslide (PCLS) Piezometer Measurements

Santa Clara Valley Water District

12/00

Piezometers P-1 & P-2
Boring No. 1-PZ; Ground Surface Elevation = **401.1'**
Installed: September 1972, by Geo-Testing, Inc.

Figure B-3



Santa Clara Valley Water District

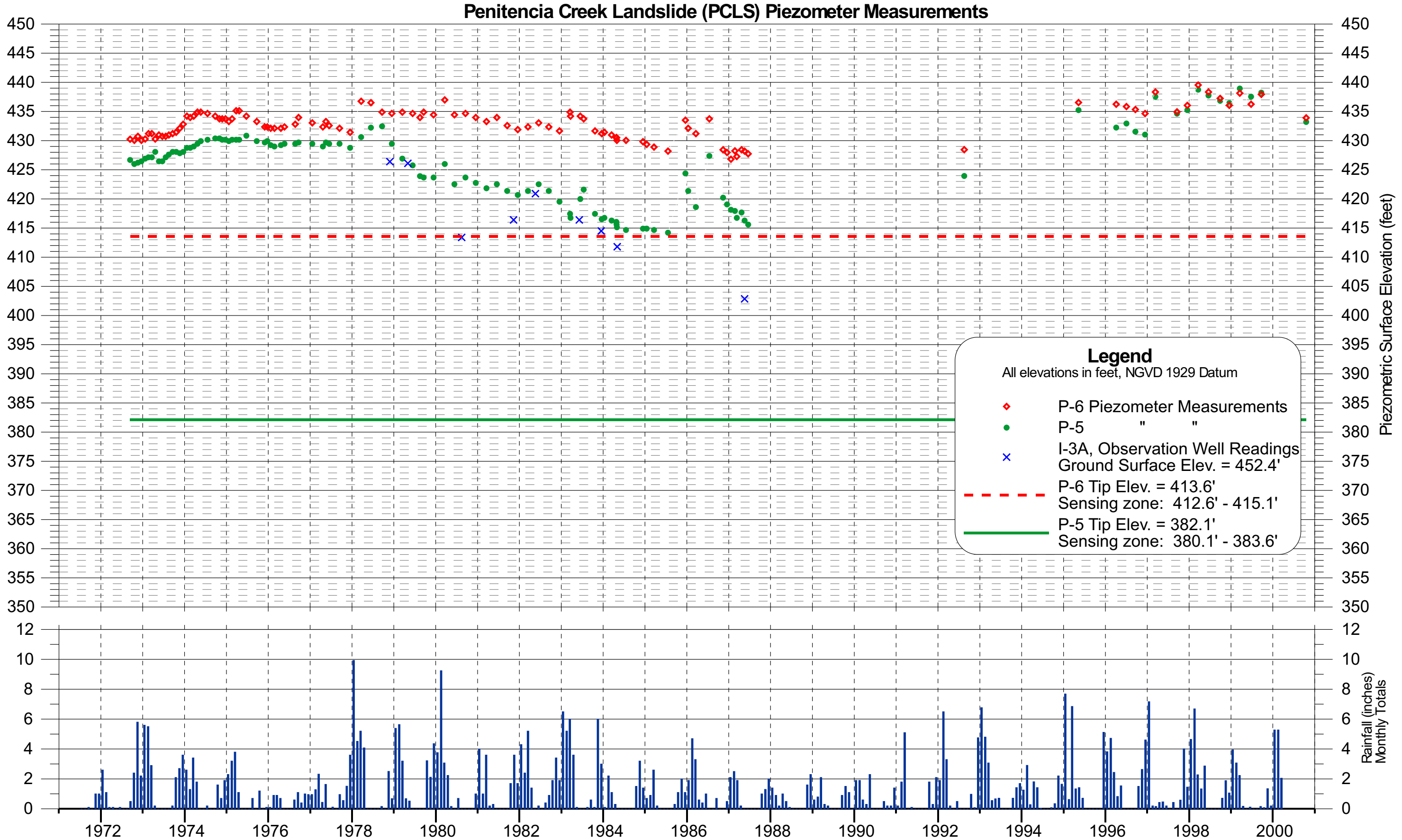
12/00

Piezometers P-3 & P-4

Boring No. 2-PZ; Ground Surface Elevation = **349.5'**

Installed: September 1972, by Geo-Testing, Inc.

Figure B-4



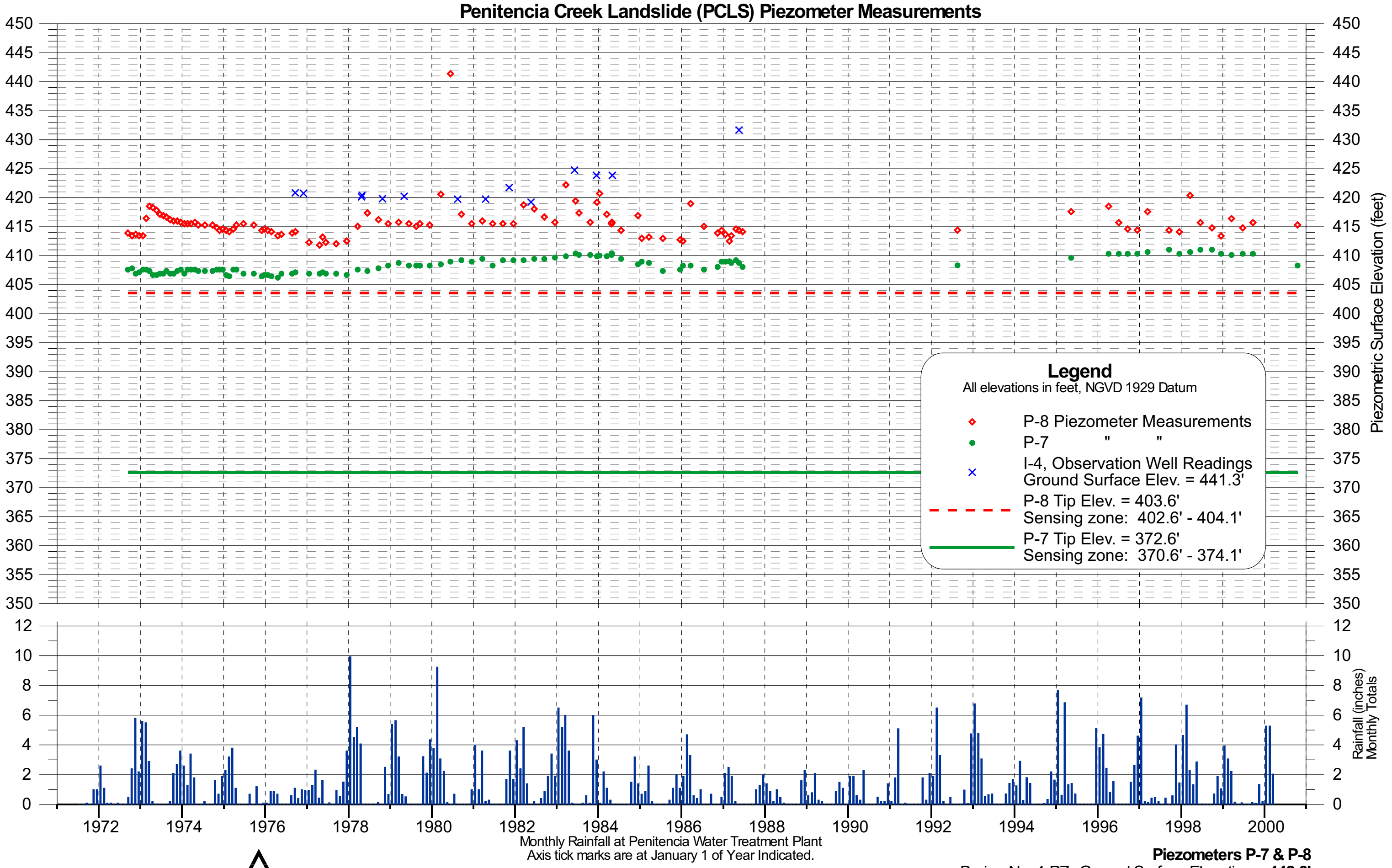
Monthly Rainfall at Penitencia Water Treatment Plant
Axis tick marks are at January 1 of Year Indicated.

Santa Clara Valley Water District

12/00

Piezometers P-5 & P-6
Boring No. 3-PZ; Ground Surface Elevation = **452.1'**
Installed: September 1972, by Geo-Testing, Inc.

Figure B-5

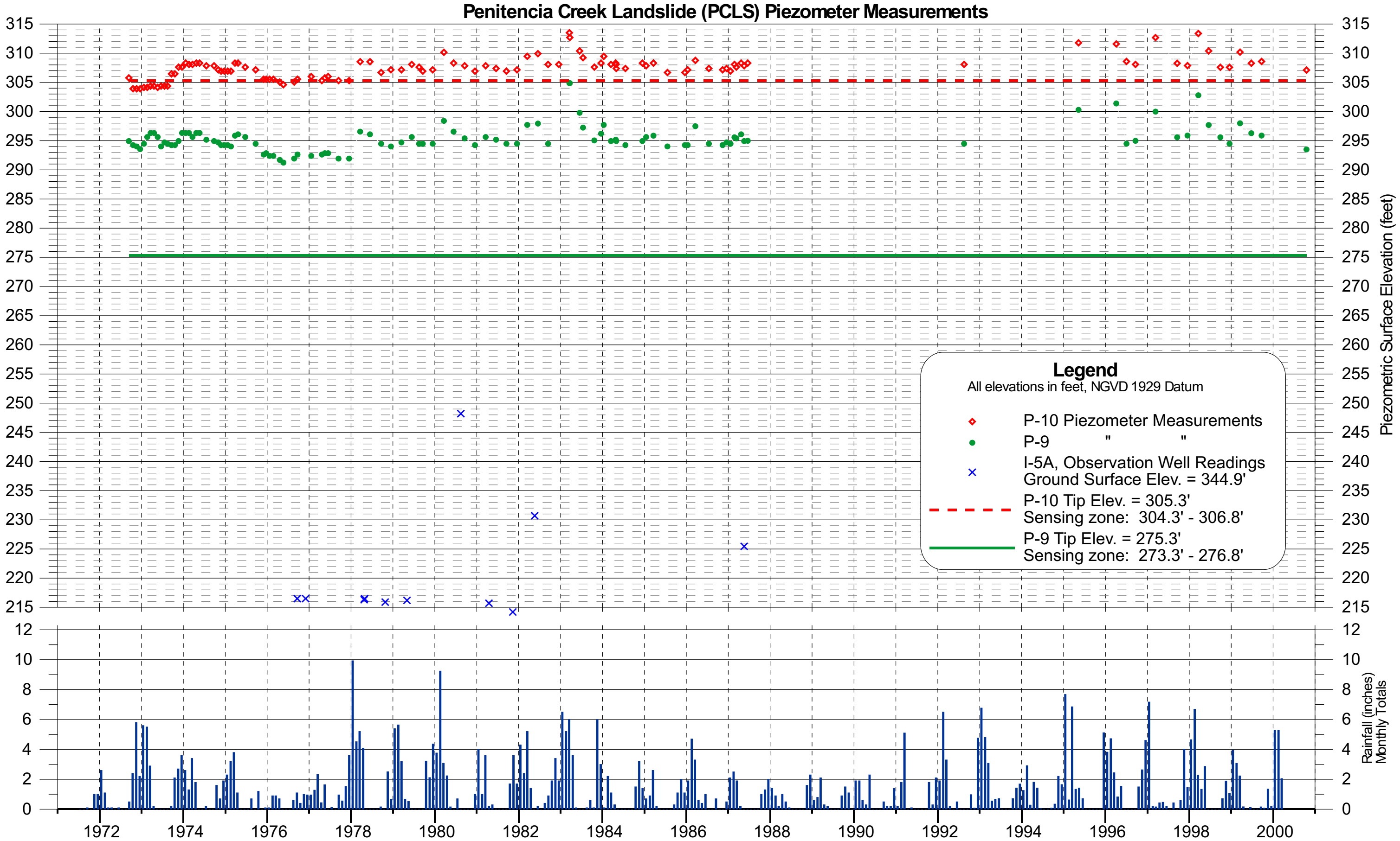


Piezometers P-7 & P-8
Boring No. 4-PZ; Ground Surface Elevation = **442.6'**
Installed: September 1972, by Geo-Testing, Inc.

Santa Clara Valley Water District
12/00

Figure B-6

Penitencia Creek Landslide (PCLS) Piezometer Measurements



Monthly Rainfall at Penitencia Water Treatment Plant
Axis tick marks are at January 1 of Year Indicated.

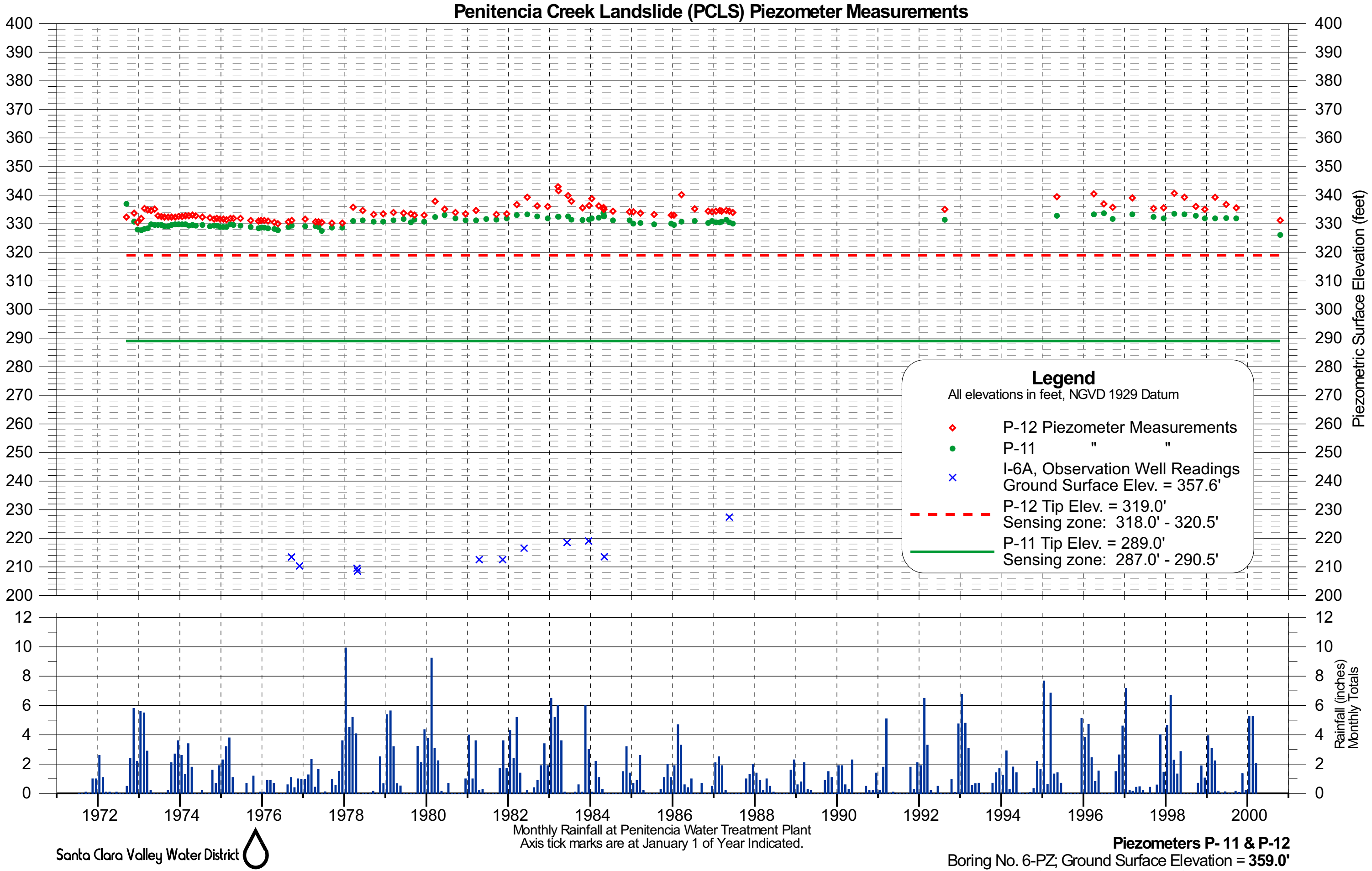
Santa Clara Valley Water District

12/00

Figure B-7

Piezometers P-9 & P-10
Boring No. 5-PZ; Ground Surface Elevation = **345.3'**
Installed: September 1972, by Geo-Testing, Inc.

Penitencia Creek Landslide (PCLS) Piezometer Measurements



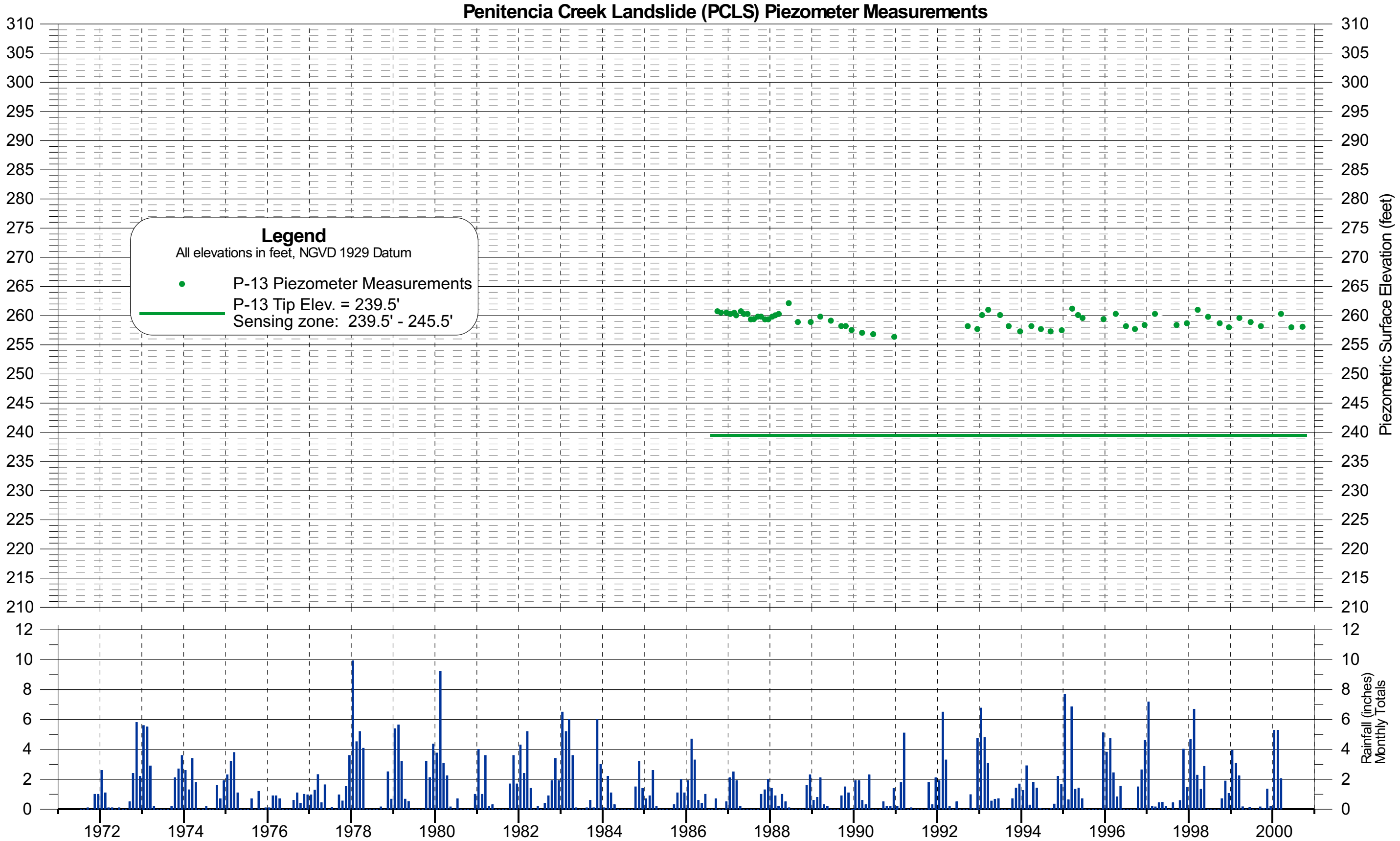
Santa Clara Valley Water District

12/00

Piezometers P- 11 & P-12
Boring No. 6-PZ; Ground Surface Elevation = **359.0'**
Installed: September 1972, by Geo-Testing, Inc.

Figure B-8

Penitencia Creek Landslide (PCLS) Piezometer Measurements



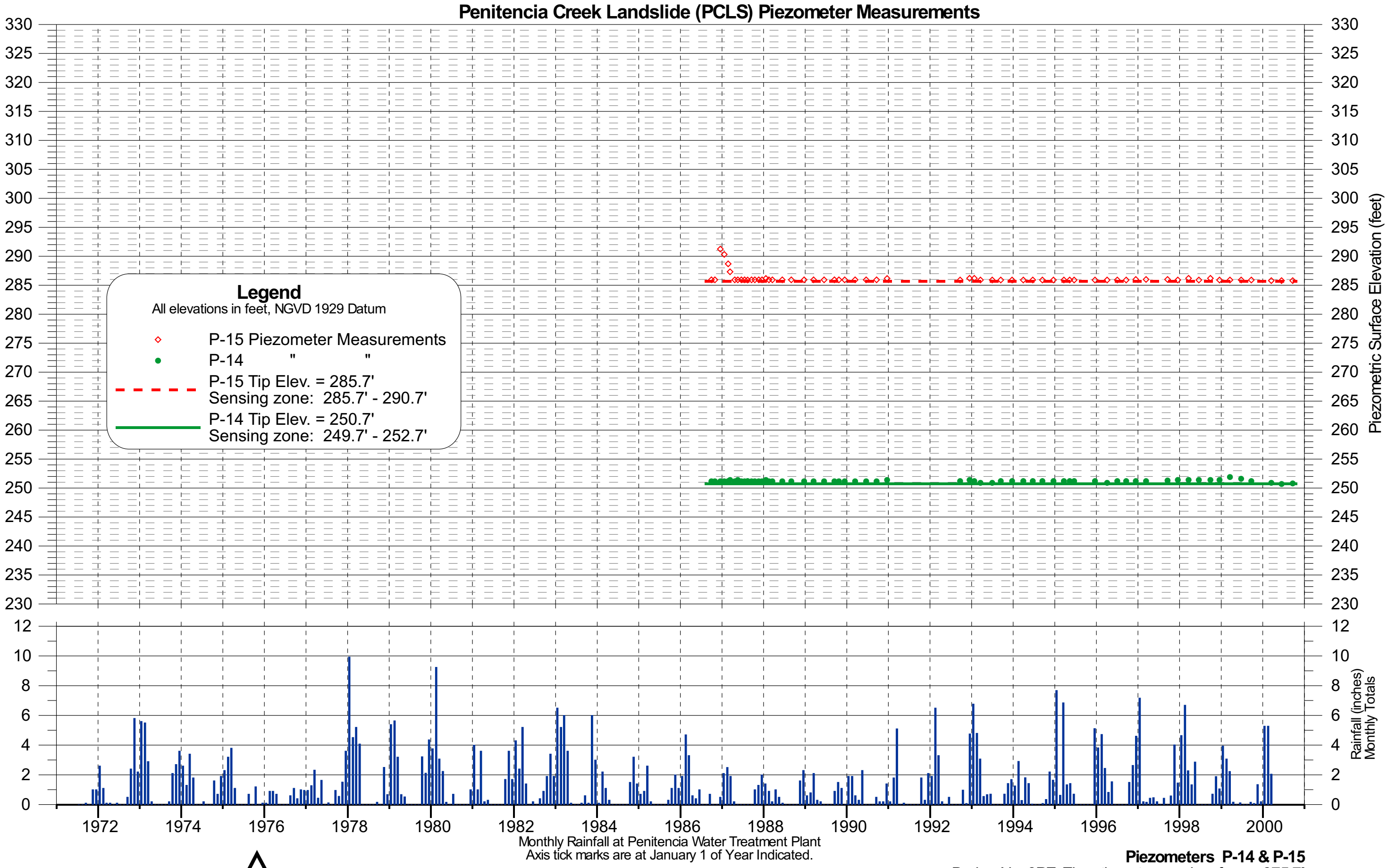
Monthly Rainfall at Penitencia Water Treatment Plant
Axis tick marks are at January 1 of Year Indicated.

Santa Clara Valley Water District

12/00

Piezometer P-13
Boring No. 7PZ; Elevation at ground surface = **284.5'**
Installed: August 1986, Pitcher Drilling Co.

Figure B-9

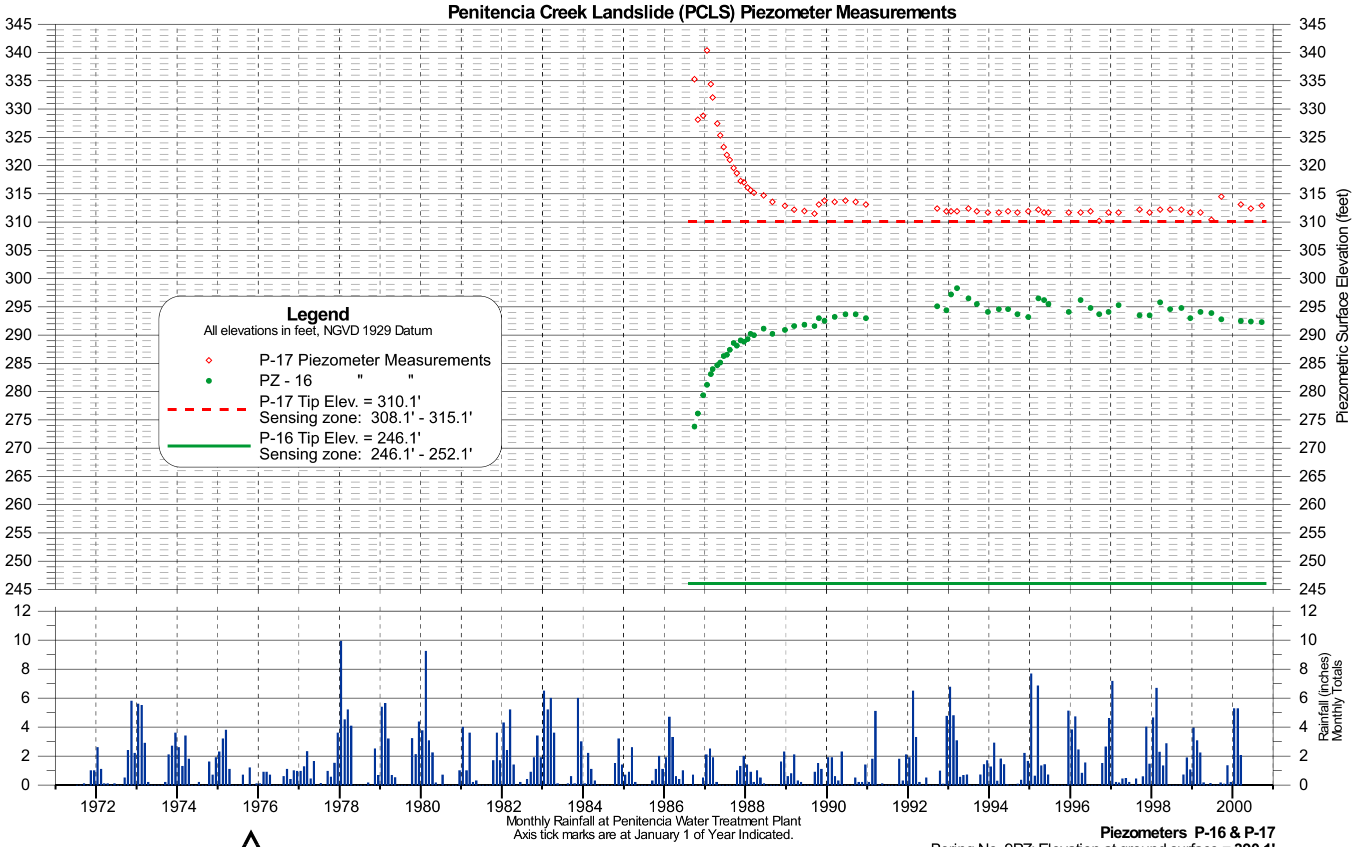


Piezometers P-14 & P-15
Boring No. 8PZ; Elevation at ground surface = **375.7'**
Installed: August 1986 by Pitcher Drilling Co.

Santa Clara Valley Water District

12/00

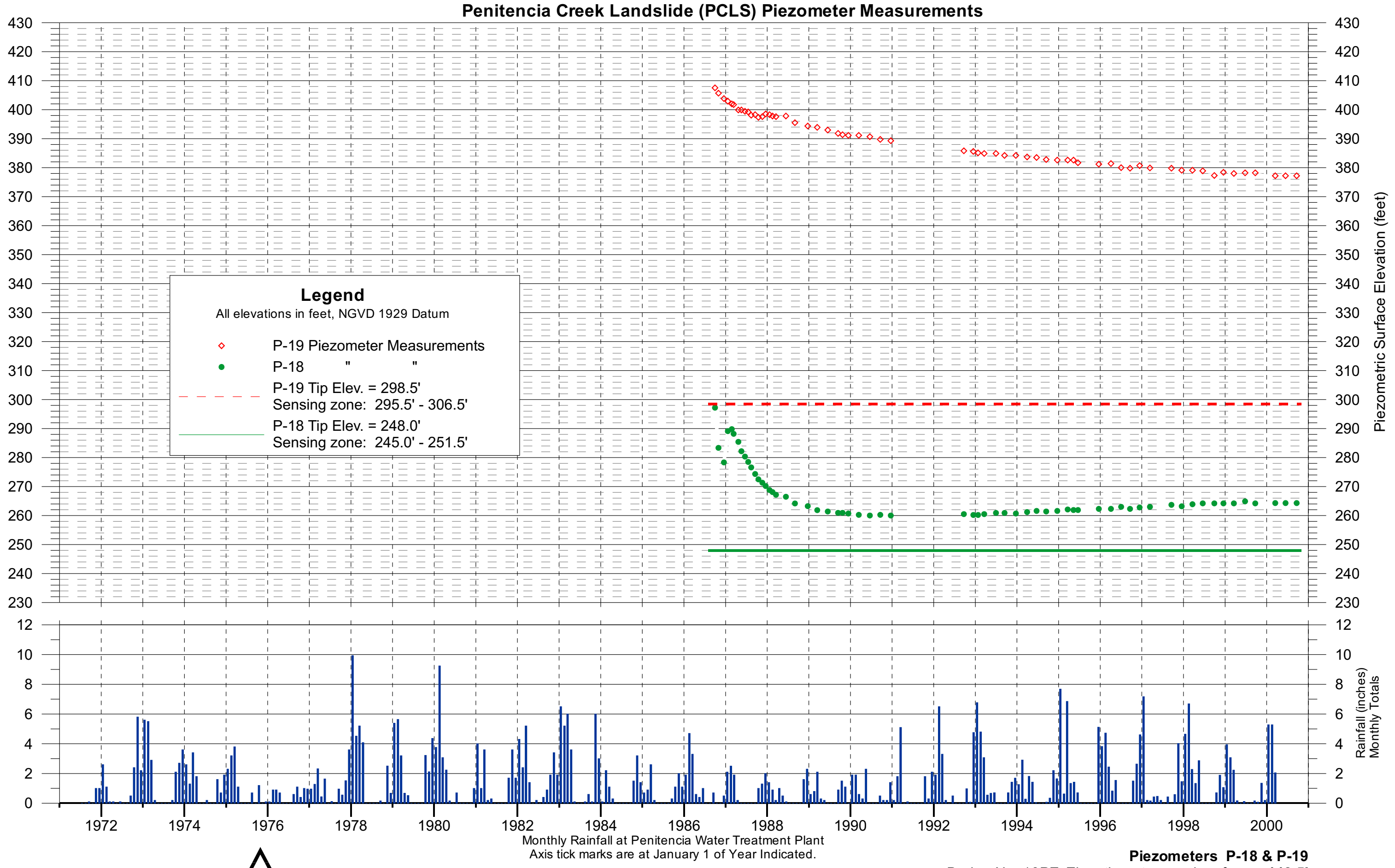
Figure B-10



Piezometers P-16 & P-17
Boring No. 9PZ; Elevation at ground surface = **390.1'**
Installed: August 1986 by Pitcher Drilling Co.

Santa Clara Valley Water District
12/00

Figure B-11


Penitencia Creek Landslide (PCLS) Piezometer Measurements

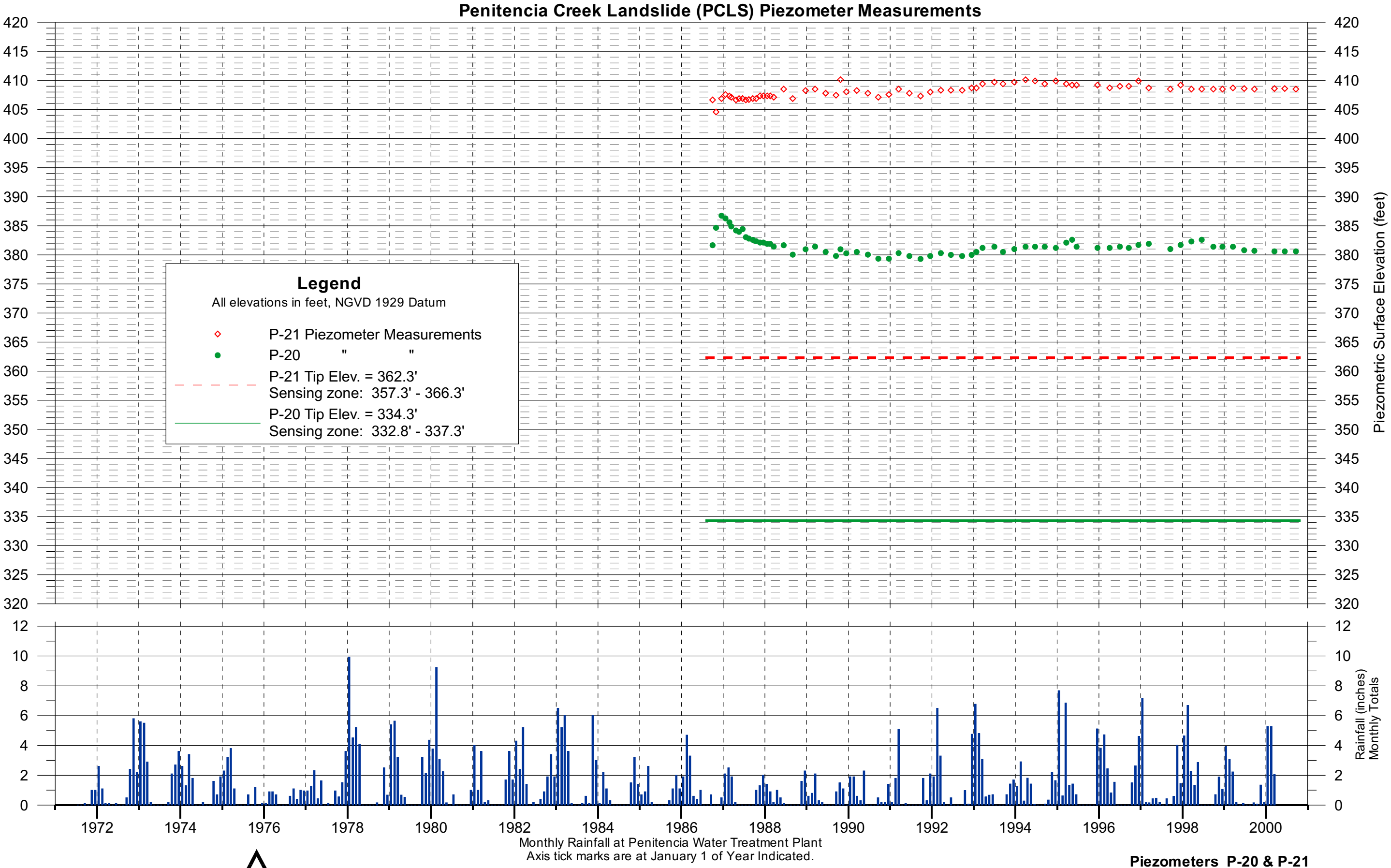
Santa Clara Valley Water District

12/00

Piezometers P-18 & P-19
Boring No. 10PZ; Elevation at ground surface = **443.5'**
Installed: August 1986 by Pitcher Drilling Co.

Figure B-12

Penitencia Creek Landslide (PCLS) Piezometer Measurements



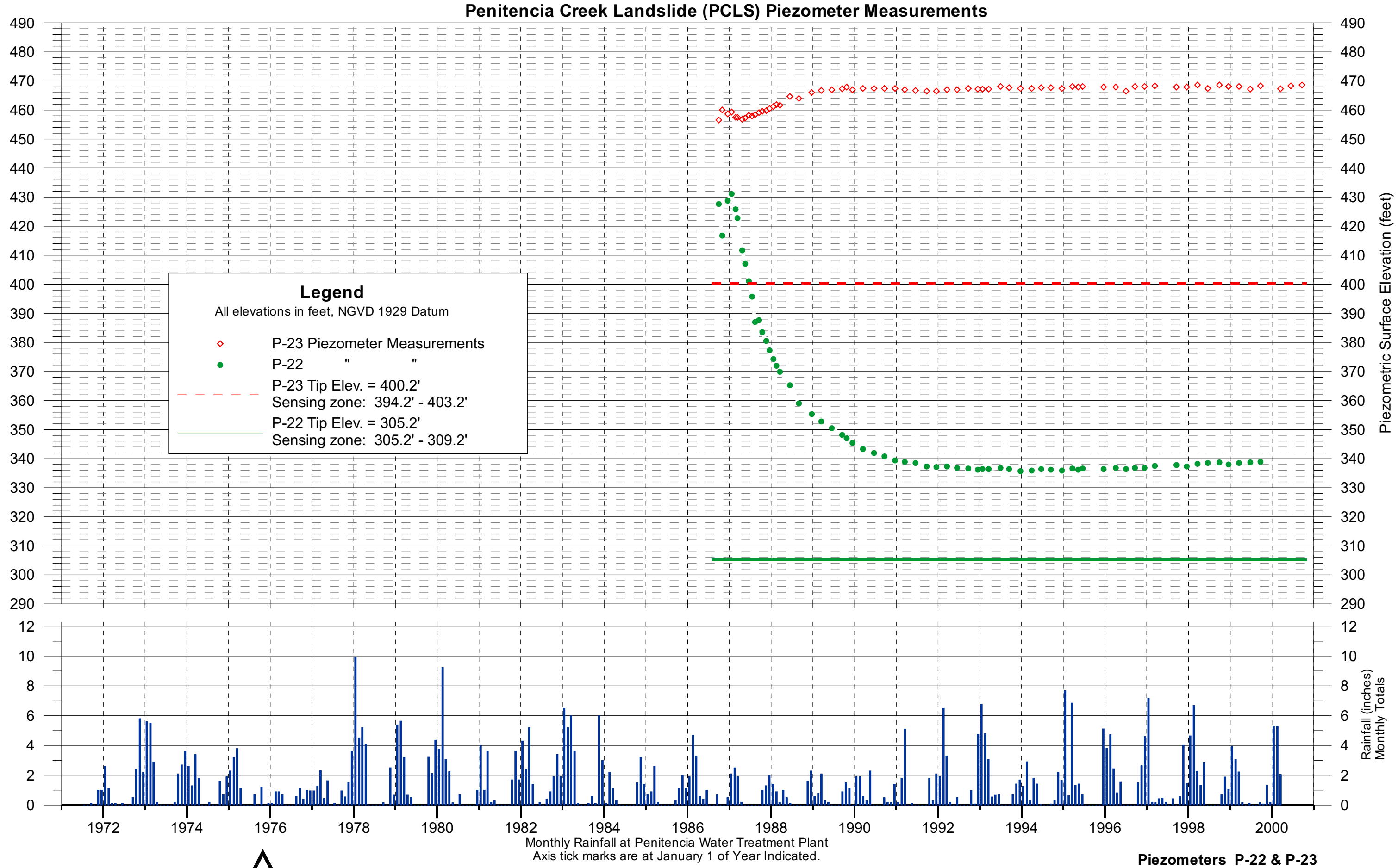
Santa Clara Valley Water District

12/00

Figure B-13

Piezometers P-20 & P-21
Boring No. 11PZ; Elevation at ground surface = **449.3'**
Installed: August 1986 by Pitcher Drilling Co.

Penitencia Creek Landslide (PCLS) Piezometer Measurements

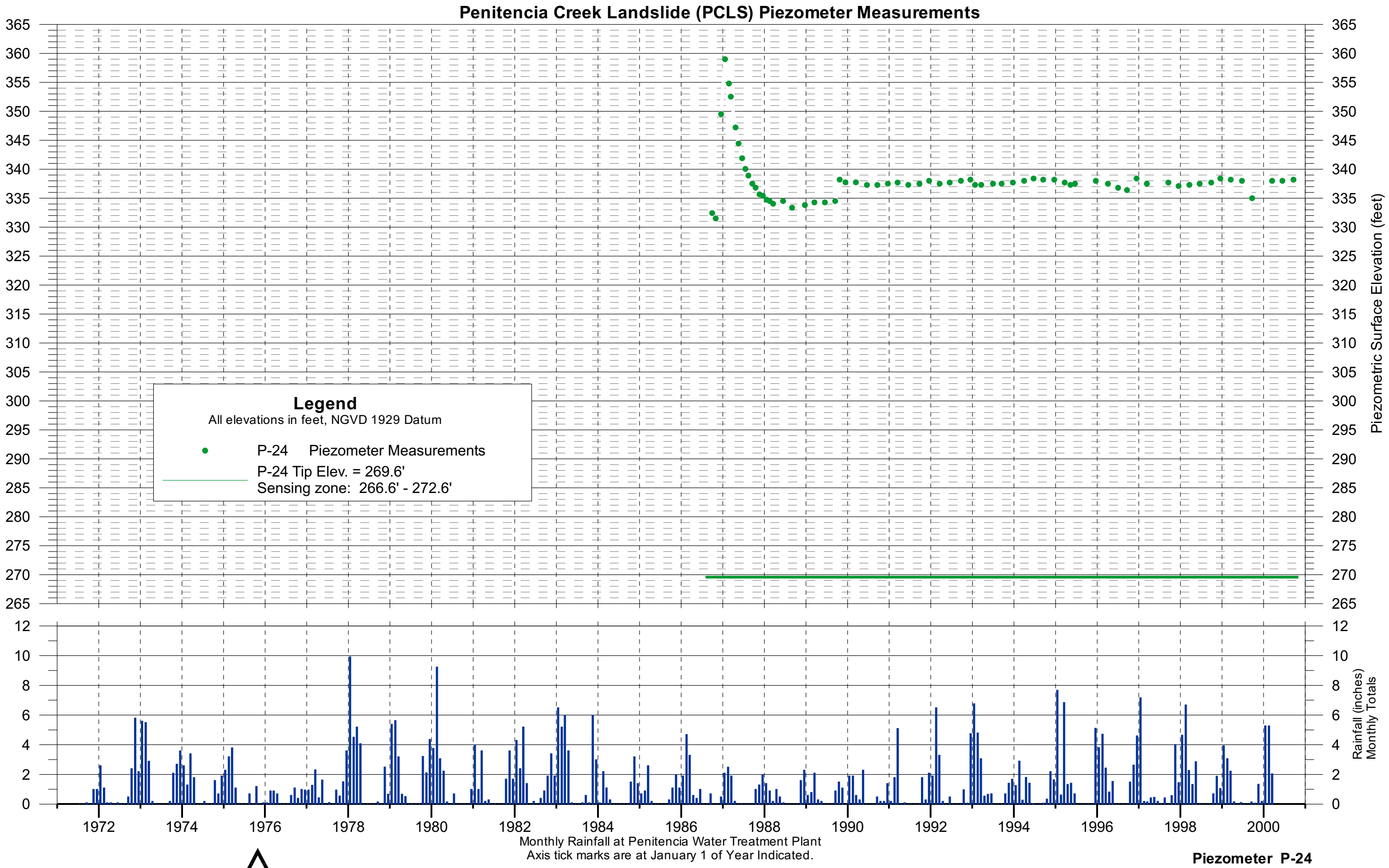


Piezometers P-22 & P-23
Boring No. 12PZ; Elevation at ground surface = **535.2'**
Installed: July 1986 by Pitcher Drilling Co.

Santa Clara Valley Water District

12/00

Figure B-14



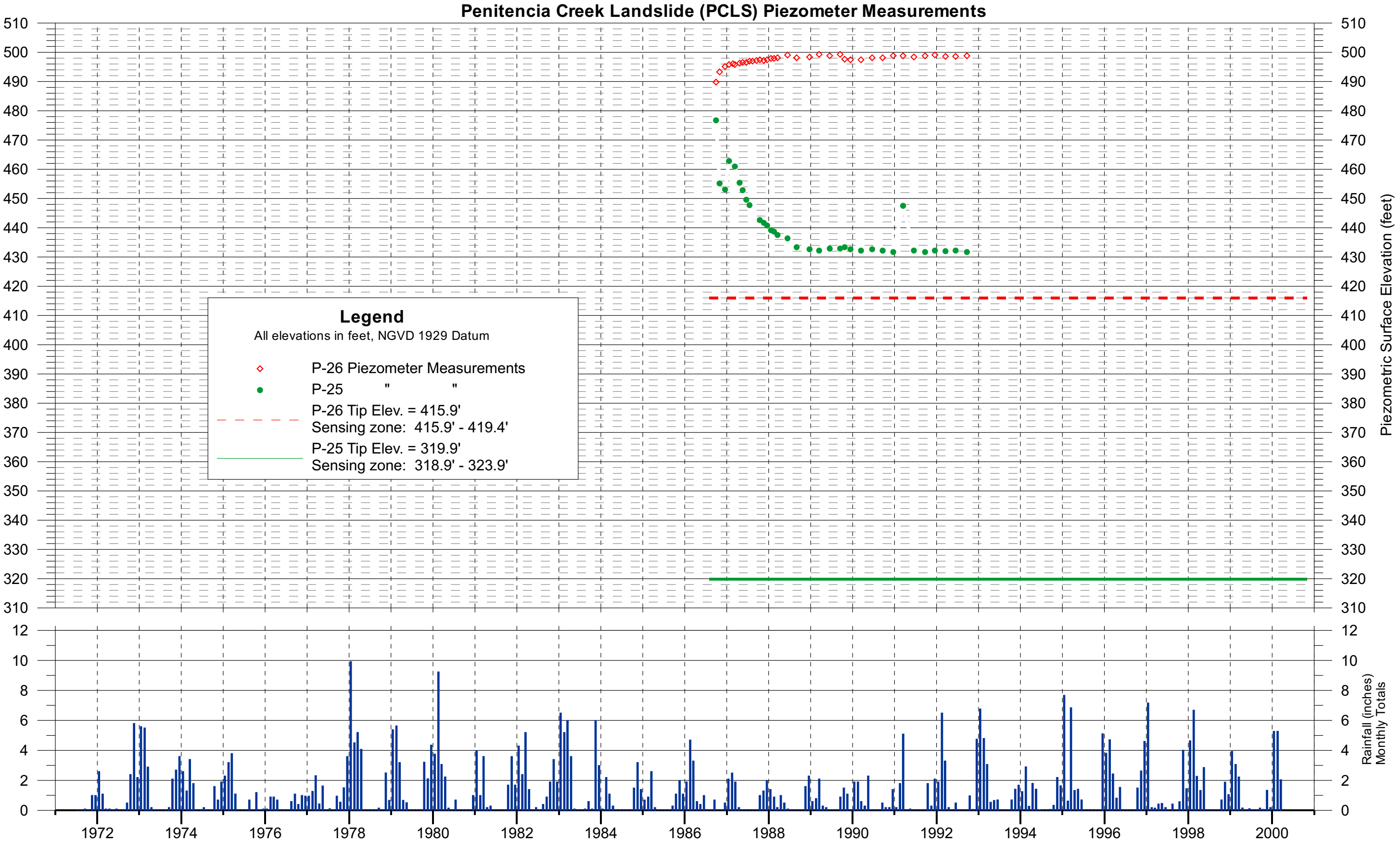
Piezometer P-24
Boring No. 13PZ; Elevation at ground surface = **414.6'**
Installed: August 1986, Pitcher Drilling Co.

Santa Clara Valley Water District

12/00

Figure B-15

Penitencia Creek Landslide (PCLS) Piezometer Measurements



Monthly Rainfall at Penitencia Water Treatment Plant
Axis tick marks are at January 1 of Year Indicated.

Santa Clara Valley Water District

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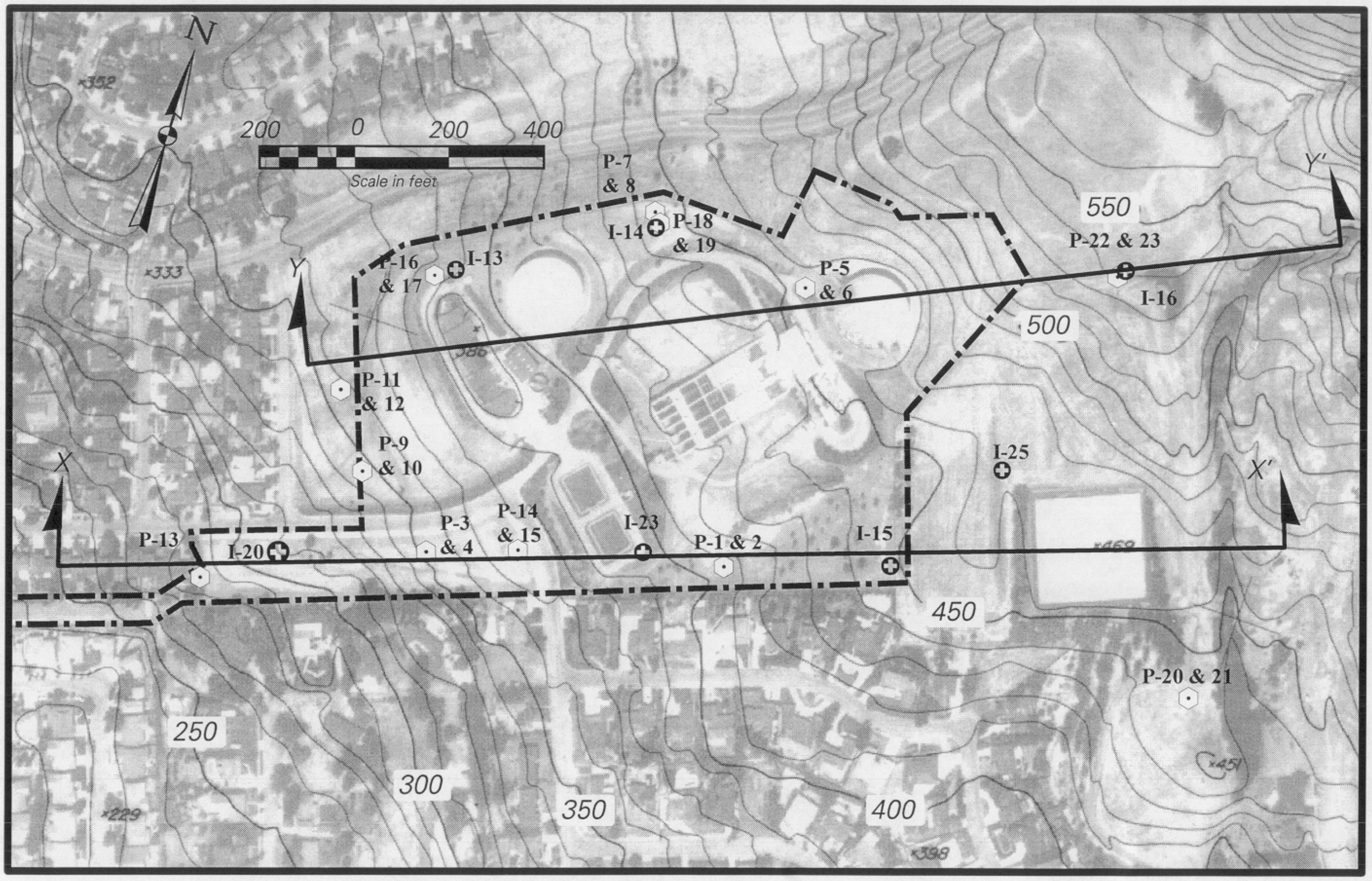
Piezometers P-25 & P-26
Boring No. 14PZ; Elevation at ground surface = **549.8'**
Installed: July 1986 by Pitcher Drilling Co.

Figure B-16

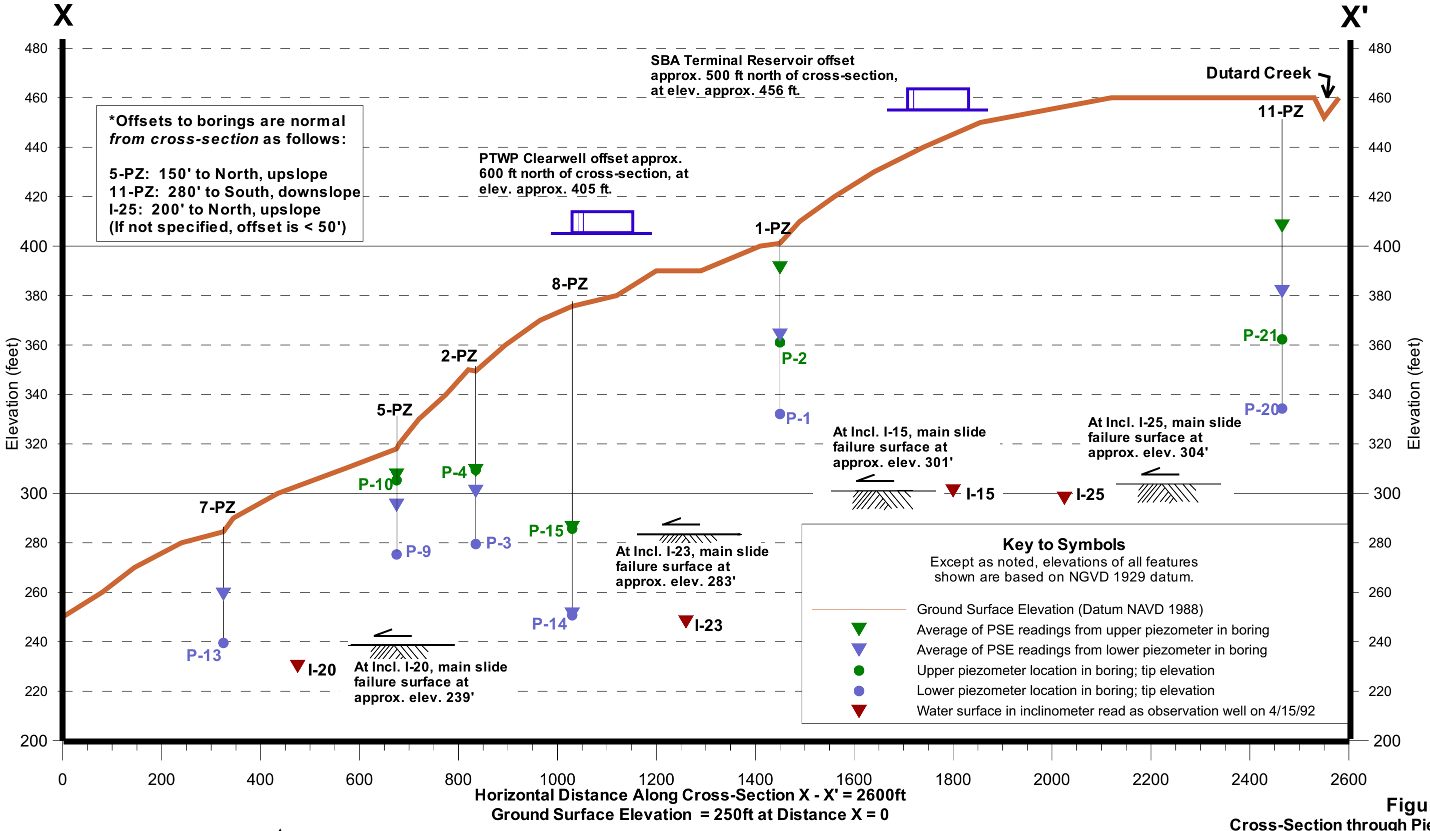


Santa Clara Valley Water District

Penitencia Creek Landslide - Location of Cross Sections and Associated Piezometers and Inclinometers

Figure B-17

Penitencia Creek Landslide Piezometric Surface Profile
Showing Representative Piezometric Surfaces (PSEs)
Cross-Section X - X' Viewing North (See Figure B-17 for Location)



Horizontal Scale: 1" = 200' Vertical Scale: 1" = 40'
(Vertical exageration = 5x)

Figure B-18.
Cross-Section through Piezometer Borings 7-PZ, 5-PZ, 2-PZ, 8-PZ, 1-PZ, and 11-PZ, and Observation Well Readings at Inclinometers I-23, I-20, I-15, I-25 occurring either on the cross-section or offset as indicated.*

Santa Clara Valley Water District

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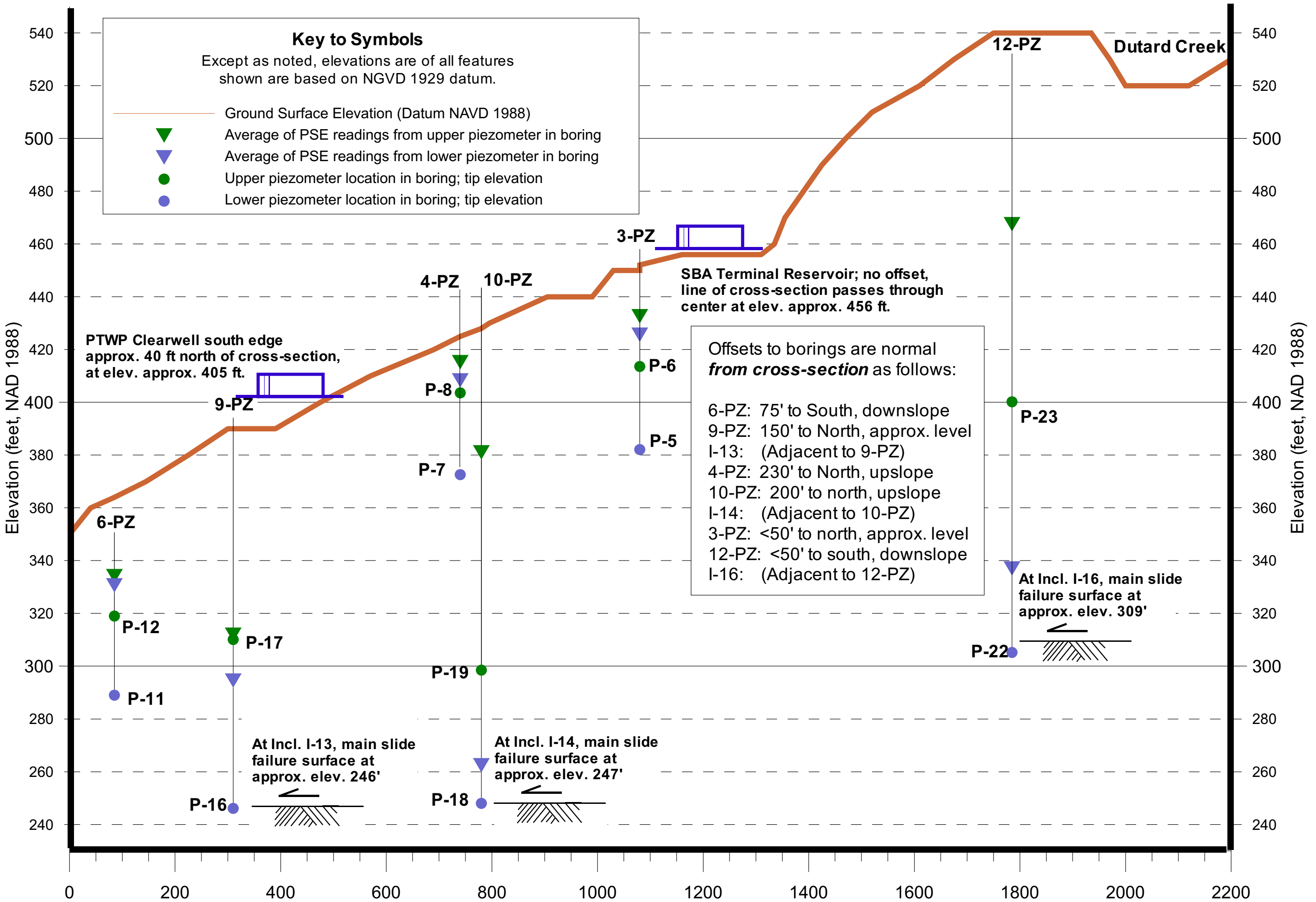
Penitencia Creek Landslide Piezometric Surface Profile
Showing Representative Piezometric Surfaces (PSEs)
Cross-Section Y-Y' Viewing North (See Figure B-17 for Location)



Key to Symbols

Except as noted, elevations are of all features
shown are based on NGVD 1929 datum.

- Ground Surface Elevation (Datum NAVD 1988)
- ▼ Average of PSE readings from upper piezometer in boring
- ▼ Average of PSE readings from lower piezometer in boring
- ● Upper piezometer location in boring; tip elevation
- ● Lower piezometer location in boring; tip elevation

PTWP Clearwell south edge approx. 40 ft north of cross-section, at elev. approx. 405 ft.

SBA Terminal Reservoir; no offset, line of cross-section passes through center at elev. approx. 456 ft.

Offsets to borings are normal *from cross-section* as follows:

6-PZ: 75' to South, downslope
9-PZ: 150' to North, approx. level
I-13: (Adjacent to 9-PZ)
4-PZ: 230' to North, upslope
10-PZ: 200' to north, upslope
I-14: (Adjacent to 10-PZ)
3-PZ: <50' to north, approx. level
12-PZ: <50' to south, downslope
I-16: (Adjacent to 12-PZ)

At Incl. I-13, main slide failure surface at approx. elev. 246'

At Incl. I-14, main slide failure surface at approx. elev. 247'

At Incl. I-16, main slide failure surface at approx. elev. 309'

Horizontal Distance Along Cross-Section X - X' = 2600ft
Ground Surface Elevation = 250ft at Distance X = 0

Horizontal Scale: 1" = 200' Vertical Scale: 1" = 40'
(Vertical exageration = 5x)

Santa Clara Valley Water District

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Figure B-19.
Cross-Section through Piezometer Borings 6-PZ, 9-PZ, 4-PZ, 10-PZ, 3-PZ, and 12-PZ, occurring left to right in cross-section shown.

APPENDIX C

DETAILED INCLINOMETER RESULTS

This appendix presents details regarding the 28 inclinometers installed for or by the District at the Penitencia Creek Landslide. A plan view showing the location of the inclinometers is presented in Figure C-1. The first series of 6 inclinometers were installed in September 1972. By September 1975, inclinometers I-2A, I-5A, and I-6A were installed to replace inclinometers I-2, I-5, and I-6 that were no longer readable at depth due to slide movement. In October 1978, inclinometer I-3A was installed to replace I-3 that had failed in April 1978. In July 1985 the District installed six new inclinometers, I-10 through I-15, and in July 1986 installed four more inclinometers, I-16 through I-19. It should be noted that inclinometer numbers I-7, I-8, and I-9 were not used in the numbering scheme. Six new inclinometers, I-20 through I-25, were installed in June 1991. Finally, in August 1997, the District installed the last two inclinometers, I-26 and I-27. At the time of this report (January 2001), only inclinometers I-26 and I-27 are being read since sufficient movement has rendered the remaining inclinometers inoperable.

Installation details for all inclinometers are summarized on Table C-1. This table also presents elevations to the top of casing and ground surface, along with elevations to the main failure surface. Individual calculated shear displacements of the main or primary shear surface for inclinometers I-10 through I-27, including H-4, are presented on Table C-2, pages 1 through 6. Individual calculated shear displacements for the secondary creep zones for those inclinometers in which such displacement occurred are presented on Table C-3, pages 1 through 5. Original plots of displacement versus depth for each inclinometer are presented in Figures C-2 through C-19 for inclinometers I-11 through I-27, including H-4. No plots are shown for the earlier inclinometers that were installed before I-10 in 1985 since the District does not have that data. Inclinometer I-10 failed within a year of having been installed and is not shown. Typically, readings of the inclinometer casing were continued in the upper part of the casing even after the instrument could not be lowered past the failure plane. By using these readings, it has been possible to interpret the location, magnitude and orientation of secondary creep movements. These secondary creep movement calculations are presented on Table C-3.

Table C-1. Pertinent Inclinometer Boring Data
Penitencia Water Treatment Plant

Inclinometer Number	Original Depth (feet)	Elevation TOC/surface (feet)	Data Coverage	A+ Direction	Primary Slip Depth (feet)	Primary Slip Elevation (feet)	Landslide Slip Dir.	Secondary Creep Depth (feet)
I-1	98.5	403.69/ 402.41	9/72 - 4/96	S77W	First 10	396	S24W	51,70
I-2	99.3	350.18/ 349.35	9/72 - 4/75	S47.5W	76	274	S31W	51
I-2A	98.9	351.03/ 350.06	9/75 - 4/96	S28W	76	275	S12W	First 11, 12.5-20, 51
I-3	248.4	454.41/ 452.40	9/72 - 4/78	S57.5W	202.5-205	249-252	S41W	First 7.5
I-3A	248	454.23/ 452.67	10/78 - 4/96	S50W	205	249		4, 80
I-4	98.5	442.75/ 441.28	9/72 - 4/96	S52W	First 7.5	<435	S56W	
I-5	248.2	346.65/ 344.86	9/72 - 4/75	S7.5E	77.5-82.5	264-269	S40W	First 10
I-5A	148.7	346.19/ 344.94	9/75 - 4/96	N87.5W	77.5-82.5	264-268	S30W	
I-6	148.9	358.52/ 357.40	9/72 - 4/75	S17W	107.5-110	248-251	S40W	
I-6A	150	358.56/ 357.59	9/75 - 4/96	S17.5W	107.5-110	249-251	S25W	
I-10	97.9	287.18/ 285.46	9/85 - 8/86	S87W	46	241	S59W	30
I-11	144.8	378.55/ 376.34	9/85 - 11/88	S55W	98	280	S25W	84-93
I-12	197.6	406.19/ 404.51	9/85 - 8/89	N87W	115.5-119.5	287-291	S28W	
I-13	174.7	392.88/ 390.61	9/85 - 11/88	S66W	147.5-150	243-245	S26W	
I-14	235.7	444.35/ 442.45	9/85 - 4/89	S27W	197-200	244-247	S31W	48-51
I-15	235.9	446.27/ 444.09	9/85 - 10/89	S74W	170-173	273-276	S24W	28-33
I-16	237.8	536.97/ 535.18	7/86 - 10/91	S50W	227-229	308-310	S50W	25, 32-35.5
I-17	195.9	451.10/ 449.05	8/86 - 4/94	S33W	137.5-139	312-314	S13W	21-23, 61-65, 91-93, 103-105.5, 111-120, 127-129.4
I-18	194.6	419.10/ 416.44	8/86 - 1/91	Due S	183-184.5	235-236	S45W	
I-19	235.6	552.95/ 550.52	9/86 - 4/96	S63W	Not encountered			160-164
I-20	95	311.24/ 308.43	6/91 - 8/95	S77W	66-71	240-245	S35W	55-57
I-21	158	370.98/ 368.78	6/91 - 7/98	S72W	123-128.5	243-248	S27W	75
I-22	258	455.53/ 453.46	6/91 - 11/99	S26W	210.8-212.5	243-245	S46W	75
I-23	158	393.26/ 391.01	6/91 - 6/00	S40W	109-111	282-284	S50W	97
I-24	260	444.30/ 441.89	6/91 - 11/99	S40W	158-160	284-286	S35W	40
I-25	236	462.9/ 460.79	6/91 - 11/99	S88W	160	303	S13W	106-109
I-26	230	438.71/ 437.05	8/97-6/00	S20W	190	249	S25W	28
I-27	210.5	436.52/ 434.72	8/97-6/00	S15W	165	272	S27W	
H-1	66	304.68/ NA	9/92-4/95	S86W	31-35	270-274	S51W	
H-2	92	270.69/ NA	9/92-6/00	S47W	None encountered			
H-3	96	284.79/ NA	9/92-	S45W	9-11	274-276	S45W	
H-4	85	688.39/ 687.34	9/92-6/00	S59W	35-45		S89W	

Note: 1) TOC = top of casing
2) File: W:\geology\penitencia creek landslide\2000 report\report\appendix C\table C-1
3) elevations based on 1929 datum

TABLE C-2
September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data
I-10 through I-27, and H-1, H-2, & H-4
Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
9/12/1985	---	0.00	0.00	0.00	0.00	0.00	---	---	---	---	---	---	---	---	---	---	---	---	---	---
9/17/1985	0.00	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/15/1986	---	---	0.10	---	---	0.09	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/14/1986	0.12	0.14	---	0.10	0.06	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/10/1986	0.99	0.72	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/14/1986	---	---	0.75	0.75	0.80	0.57	---	---	---	---	---	---	---	---	---	---	---	---	---	---
7/24/1986	---	---	---	---	---	---	0.00	---	---	---	---	---	---	---	---	---	---	---	---	---
8/4/1986	---	---	---	---	---	---	---	0.00	---	---	---	---	---	---	---	---	---	---	---	---
8/14/1986	1.21	0.90	0.75	0.96	1.01	0.69	---	---	0.00	---	---	---	---	---	---	---	---	---	---	---
11/12/1986	---	1.03	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/14/1986	---	---	0.85	1.01	1.08	0.77	---	---	---	---	---	---	---	---	---	---	---	---	---	---
12/24/1986	---	1.04	---	1.03	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
12/31/1986	---	1.06	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/4/1987	---	---	---	1.18	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/7/1987	---	1.13	---	1.14	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/14/1987	---	---	0.99	---	1.28	0.89	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/15/1987	---	---	---	---	---	---	0.12	---	---	---	---	---	---	---	---	---	---	---	---	---
1/21/1987	---	---	---	1.12	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/28/1987	---	---	---	1.18	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
2/4/1987	---	1.15	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
3/4/1987	---	1.16	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/9/1987	---	1.16	0.98	1.18	1.28	0.89	---	---	---	---	---	---	---	---	---	---	---	---	---	---

TABLE C-2
September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data
I-10 through I-27, and H-1, H-2, & H-4
Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
4/13/1987	---	---	---	---	---	---	0.13	---	---	---	---	---	---	---	---	---	---	---	---	---
8/17/1987	---	---	---	---	---	0.96	---	0.09	---	---	---	---	---	---	---	---	---	---	---	---
8/18/1987	---	1.24	1.14	1.20	1.38	---	0.21	---	0.10	---	---	---	---	---	---	---	---	---	---	---
11/11/1987	---	---	---	---	---	---	---	0.24	---	---	---	---	---	---	---	---	---	---	---	---
11/18/1987	---	1.36	---	1.32	1.49	---	---	---	0.24	---	---	---	---	---	---	---	---	---	---	---
11/19/1987	---	1.38	1.10	---	---	1.01	0.36	---	---	---	---	---	---	---	---	---	---	---	---	---
1/20/1988	---	---	---	1.30	1.48	1.00	0.41	---	0.20	---	---	---	---	---	---	---	---	---	---	---
1/28/1988	---	---	---	---	---	---	---	0.36	---	---	---	---	---	---	---	---	---	---	---	---
4/20/1988	---	---	---	---	1.62	---	---		---	---	---	---	---	---	---	---	---	---	---	---
4/28/1988	---	1.48	1.18	1.48	---	1.13	0.49	0.48	0.36	---	---	---	---	---	---	---	---	---	---	---
6/14/1988	---	1.52	---	1.54	---	1.14	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/31/1988	---	1.57	---	---	1.72	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
9/1/1988	---	---	1.23	---	---	1.20	0.56	0.56	---	---	---	---	---	---	---	---	---	---	---	---
9/10/1988	---	---	---	---	---	---	---	---	0.43	---	---	---	---	---	---	---	---	---	---	---
11/9/1988	---	---	---	1.74	---	---	---	---	0.54	---	---	---	---	---	---	---	---	---	---	---
11/10/1988	---	1.69	---	---	1.82	1.14	0.67	0.67	---	---	---	---	---	---	---	---	---	---	---	---
1/31/1989	---	---	1.34	---	---	1.13	0.67	0.70	---	---	---	---	---	---	---	---	---	---	---	---
2/1/1989	---	---	---	---	---	---	---	---	0.54	---	---	---	---	---	---	---	---	---	---	---
4/4/1989	---	---	---	---	1.93	---	0.79	0.82	---	---	---	---	---	---	---	---	---	---	---	---
4/5/1989	---	---	1.43	---	---	1.29	---	---	0.66	---	---	---	---	---	---	---	---	---	---	---
8/29/1989	---	---	1.55	---	---	---	0.79	0.84	---	---	---	---	---	---	---	---	---	---	---	---
8/30/1989	---	---	---	---	---	---	---	---	0.73	---	---	---	---	---	---	---	---	---	---	---

TABLE C-2

September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data

I-10 through I-27, and H-1, H-2, & H-4

Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
9/29/1989	---	---	---	---	---	1.29	---	---	---	---	---	---	---	---	---	---	---	---	---	---
10/30/1989	---	---	---	---	---	1.65	---	1.37	1.52	---	---	---	---	---	---	---	---	---	---	---
10/31/1989	---	---	---	---	---	---	1.36	---	---	---	---	---	---	---	---	---	---	---	---	---
2/13/1990	---	---	---	---	---	---	1.46	1.43	---	---	---	---	---	---	---	---	---	---	---	---
2/14/1990	---	---	---	---	---	---	---	---	1.63	---	---	---	---	---	---	---	---	---	---	---
5/14/1990	---	---	---	---	---	---	1.52	1.43		---	---	---	---	---	---	---	---	---	---	---
5/15/1990	---	---	---	---	---	---	---	---	1.72	---	---	---	---	---	---	---	---	---	---	---
9/5/1990	---	---	---	---	---	---	---	---	1.76	---	---	---	---	---	---	---	---	---	---	---
9/6/1990	---	---	---	---	---	---	1.55	1.49	---	---	---	---	---	---	---	---	---	---	---	---
1/3/1991	---	---	---	---	---	---	---	---	1.94	---	---	---	---	---	---	---	---	---	---	---
1/4/1991	---	---	---	---	---	---	1.58	1.46	---	---	---	---	---	---	---	---	---	---	---	---
4/16/1991	---	---	---	---	---	---	1.60	1.50	---	---	---	---	---	---	---	---	---	---	---	---
6/12/1991	---	---	---	---	---	---	---	---	---	0.00	0.00	0.00	---	---	---	---	---	---	---	---
10/25/1991	---	---	---	---	---	---	1.58	1.47	---	---	---	---	---	---	---	---	---	---	---	---
11/14/1991	---	---	---	---	---	---	---	---	---	---	0.08	---	---	---	---	---	---	---	---	---
11/18/1991	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/19/1991	---	---	---	---	---	---	---	---	---	---	---	0.16	0.08	0.11	---	---	---	---	---	---
4/15/1992	---	---	---	---	---	---	---	---	---	---	0.20	---	0.12	0.13	---	---	---	---	---	---
4/16/1992	---	---	---	---	---	---	---	---	---	0.30	---	---	---	---	---	---	---	---	---	---
4/22/1992	---	---	---	---	---	---	---	1.53	---	---	---	0.18	---	---	0.13	---	---	---	---	---
8/11/1992	---	---	---	---	---	---	---	---	---	0.40	0.27	---	0.12	0.14	---	---	---	---	---	---
8/12/1992	---	---	---	---	---	---	---	1.52	---	---	---	0.20	---	---	---	---	---	---	---	---

TABLE C-2
September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data
I-10 through I-27, and H-1, H-2, & H-4
Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
11/23/1992	---	---	---	---	---	---	---	1.52	---	---	0.22	---	---	---	---	---	---	---	---	---
11/24/1992	---	---	---	---	---	---	---	---	---	0.42	---	---	0.21	---	---	---	---	---	0.00	0.00
1/25/1993	---	---	---	---	---	---	---	---	---	---	---	0.32	---	0.22	---	---	---	---	---	---
1/26/1993	---	---	---	---	---	---	---	1.57	---	---	0.39	---	---	---	0.26	---	---	0.00	0.24	---
2/25/1993	---	---	---	---	---	---	---	---	---	0.83	---	---	---	---	---	---	---	---	---	---
4/23/1993	---	---	---	---	---	---	---	1.81	---	---	0.56	---	---	0.34	---	---	---	0.22	0.64	0.14
4/27/1993	---	---	---	---	---	---	---	---	---	0.98	---	0.49	0.31	---	0.38	---	---	---	---	---
8/12/1993	---	---	---	---	---	---	---	---	---	0.99	0.58	0.47	0.33	0.3	0.39	---	---	---	---	---
8/13/1993	---	---	---	---	---	---	---	1.86	---	---	---	---	---	---	---	---	---	0.25	---	---
8/16/1993	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.61	0.18
12/9/1993	---	---	---	---	---	---	---	1.86	---	1.04	---	---	0.33	0.35	---	---	---	---	---	0.23
12/13/1993	---	---	---	---	---	---	---	---	---	---	0.57	---	---	---	0.4	---	---	0.22	0.61	---
2/24/1994	---	---	---	---	---	---	---	1.92	---	1.10	---	0.54	0.35	0.38	---	---	---	---	0.63	0.24
3/1/1994	---	---	---	---	---	---	---	---	---	---	0.67	---	---	---	0.48	---	---	0.31	---	---
4/27/1994	---	---	---	---	---	---	---	2.00	---	1.14	0.66	---	0.39	0.45	0.49	---	---	---	---	0.25
4/28/1994	---	---	---	---	---	---	---	---	---	---	---	0.60	---	---	---	---	---	0.26	0.63	---
8/18/1994	---	---	---	---	---	---	---	---	---	1.19	0.69	0.58	0.4	0.44	0.49	---	---	0.31	0.63	0.28
11/1/1994	---	---	---	---	---	---	---	---	---	1.24	0.66	0.62	0.4	0.44	0.52	---	---	0.29	0.62	0.30
2/14/1995	---	---	---	---	---	---	---	---	---	1.70	---	0.79	---	---	---	---	---	0.49	0.69	0.34
2/15/1995	---	---	---	---	---	---	---	---	---	---	0.85	---	0.52	0.58	---	---	---	---	---	---
4/18/1995	---	---	---	---	---	---	---	---	---	1.85	0.96	0.82	0.58	0.64	0.72	---	---	0.87	1.42	0.39
8/26/1995	---	---	---	---	---	---	---	---	---	1.83	---	0.84	0.6	0.65	---	---	---	---	1.42	0.45

TABLE C-2

September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data

I-10 through I-27, and H-1, H-2, & H-4

Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
11/13/1995	---	---	---	---	---	---	---	---	---	---	---	---	0.62	---	---	---	---	---	1.38	---
11/14/1995	---	---	---	---	---	---	---	---	---	---	1.04	1.02	---	0.73	0.83	---	---	---	---	---
4/10/1996	---	---	---	---	---	---	---	---	---	---	---	---	0.71	---	---	---	---	---	1.98	0.51
4/16/1996	---	---	---	---	---	---	---	---	---	---	1.14	1.14	---	0.81	0.95	---	---	---	---	---
5/28/1996	---	---	---	---	---	---	---	---	---	---	1.18	1.15	0.71	---	---	---	---	---	---	---
5/29/1996	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.95	---	---	---	---	---
8/23/1996	---	---	---	---	---	---	---	---	---	---	---	---	0.72	0.82	0.99	---	---	---	2.01	0.52
8/26/1996	---	---	---	---	---	---	---	---	---	---	1.19	1.10	---	---	---	---	---	---	---	---
9/3/1996	---	---	---	---	---	---	---	---	---	---	---	---	0.72	0.79	0.99	---	---	---	---	---
11/7/1996	---	---	---	---	---	---	---	---	---	---	---	---	0.72	---	---	---	---	---	---	0.53
11/12/1996	---	---	---	---	---	---	---	---	---	---	1.18	---	---	0.87	1	---	---	---	---	---
2/6/1997	---	---	---	---	---	---	---	---	---	---	1.68	1.37	0.87	1.01	1.18	---	---	---	2.81	---
6/12/1997	---	---	---	---	---	---	---	---	---	---	1.70	1.36	0.9	1	1.23	---	---	---	---	---
8/6/1997	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0	---	---	---	---
8/11/1997	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0	---	---	
9/15/1997	---	---	---	---	---	---	---	---	---	---	1.78	1.51	---	1.09	1.36	---	---	---	---	---
9/16/1997	---	---	---	---	---	---	---	---	---	---	---	---	0.96	---	---	---	---	---	2.81	0.63
11/25/1997	---	---	---	---	---	---	---	---	---	---	---	---	0.96	1.09	1.38	---	---	---	---	
12/1/1997	---	---	---	---	---	---	---	---	---	---	---	1.48	---	---	---	---	---	---	---	---
12/2/1997	---	---	---	---	---	---	---	---	---	---	1.79	---	---	---	---	---	---	---	---	---
12/3/1997	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.64
3/2/1998	---	---	---	---	---	---	---	---	---	---	2.08	1.68	1.09	---	1.62	0.22	0.18	---	---	---

TABLE C-2

September 1985 through June 2000 Inclinometer Main Shear Surface Displacement Data
I-10 through I-27, and H-1, H-2, & H-4
Penitencia Water Treatement Plant

Date	Inclinometer Number																			
	I-10	I-11	I-12	I-13	I-14	I-15	I-16	I-17	I-18	I20	I-21	I-22	I-23	I-24	I-25	I-26	I-27	H-1	H-3	H-4
	Displacement in inches																			
3/3/1998	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.77
6/22/1998	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.87
6/26/1998	---	---	---	---	---	---	---	---	---	---	---	---	1.14	1.32	---	---	---	---	---	---
7/13/1998	---	---	---	---	---	---	---	---	---	---	2.17	---	---	---	---	0.23	0.205	---	---	---
7/14/1998	---	---	---	---	---	---	---	---	---	---	---	1.73	---	---	---	---	---	---	---	---
9/28/1998	---	---	---	---	---	---	---	---	---	---	---	---	1.14	1.34	---	0.25	0.21	---	---	0.87
9/29/1998	---	---	---	---	---	---	---	---	---	---	---	1.72	---	---	---	---	---	---	---	---
12/17/1998	---	---	---	---	---	---	---	---	---	---	---	---	1.18	---	---	0.27	---	---	---	---
12/19/1998	---	---	---	---	---	---	---	---	---	---	---	1.72	---	1.36	1.6	---	0.22	---	---	0.86
2/19/1999	---	---	---	---	---	---	---	---	---	---	---	---	1.21	1.54	---	---	---	---	---	---
2/23/1999	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.95
2/24/1999	---	---	---	---	---	---	---	---	---	---	---	---	---	---	1.72	---	0.32	---	---	---
3/1/1999	---	---	---	---	---	---	---	---	---	---	---	1.89	---	---	---	0.38	---	---	---	---
5/27/1999	---	---	---	---	---	---	---	---	---	---	2.43	---	1.26	---	---	0.43	---	---	---	---
6/2/1999	---	---	---	---	---	---	---	---	---	---	---	1.93	---	1.61	---	---	---	---	---	---
8/24/1999	---	---	---	---	---	---	---	---	---	---	---	---	1.29	---	---	---	---	---	---	---
8/26/1999	---	---	---	---	---	---	---	---	---	---	2.43	1.89	---	---	1.68	---	---	---	---	0.98
11/13/1999	---	---	---	---	---	---	---	---	---	---	2.57	---	1.34	1.68	1.97	0.46	0.4	---	---	---
11/15/1999	---		---	---	---	---	---	---	---	---	---	1.98	---	---	---	---	---	---	---	0.96
3/7/2000	---	---	---	---	---	---	---	---	---	---	---	---	1.39	---	---	---	---	---	---	---
3/8/2000	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.56	0.48	---	---	---
3/11/2000	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.99
6/12/2000	---	---	---	---	---	---	---	---	---	---	---	---	1.45	---	---	---	---	---	---	---
6/13/2000	---	---	---	---	---	---	---	---	---	---	---	---	---	---	---	0.62	0.54	---	---	1.05

Table C-3
April 1986 through November 1999 Inclinometer Secondary Creep Zone Displacement Data
Inclinometers I-11, I-14 through I-27
Penitencia Water Treatment Plant

Date	Inclinometer No	I-11	I-14	I-15	I-16	I-16	I-17	I-19	I-20	I-21	I-22	I-23	I-24	I-25	I-26	I-27
	Depth in feet	84-93	48-51	28-33	32-35.5	25	(6 zones*)	160-164	55-57	75	75	97	40	106-109	28	
		Displacement in inches														
04/10/86		0.07	---	---	---	---	---	---	---	---	---	---	---	---	---	---
08/14/86		0.08	---	---	---	---	---	---	---	---	---	---	---	---	---	---
12/24/86		0.09	---	---	---	---	---	---	---	---	---	---	---	---	---	---
03/04/87		0.11	---	---	---	---	---	---	---	---	---	---	---	---	---	---
08/18/87		---	---		0.07	0.07	---	0.01	---	---	---	---	---	---	---	---
11/18/87		0.13	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/19/87		---	---	---	0.08	0.06	---	---	---	---	---	---	---	---	---	---
01/20/88		---	---	---	0.14	0.1	---	0.02	---	---	---	---	---	---	---	---
04/28/88		0.13	---	---	0.15	0.07	---	---	---	---	---	---	---	---	---	---
08/31/88		0.15	---	---	---	---	---	---	---	---	---	---	---	---	---	---
09/01/88		---	---	---	0.19	0.14	---	0.01	---	---	---	---	---	---	---	---
11/10/88		---	---	---	0.23	0.15	---	---	---	---	---	---	---	---	---	---
11/15/88		---	---	---			---	0.02	---	---	---	---	---	---	---	---
01/31/89		---	---	---	0.21	0.14	---	---	---	---	---	---	---	---	---	---
02/01/89		---	---	---			---	0.01	---	---	---	---	---	---	---	---
04/04/89		---	---	---	0.24	0.17	---	---	---	---	---	---	---	---	---	---
04/05/89		---	---	---			---	0.02	---	---	---	---	---	---	---	---
08/29/89		---	---	---	0.21	0.16	---	---	---	---	---	---	---	---	---	---
08/30/89		---	---	---	---	---	---	0.02	---	---	---	---	---	---	---	---
10/30/89		---	0.04	---	---	---	---	---	---	---	---	---	---	---	---	---
10/31/89		---	---	---	0.6	0.27	---	0.09	---	---	---	---	---	---	---	---

Table C-3
April 1986 through November 1999 Inclinometer Secondary Creep Zone Displacement Data
Inclinometers I-11, I-14 through I-27
Penitencia Water Treatment Plant

Date	Inclinometer No	I-11	I-14	I-15	I-16	I-16	I-17	I-19	I-20	I-21	I-22	I-23	I-24	I-25	I-26	I-27
	Depth in feet	84-93	48-51	28-33	32-35.5	25	(6 zones*)	160-164	55-57	75	75	97	40	106-109	28	
		Displacement in inches														
02/13/90		---	---	---	0.62	0.25	---	---	---	---	---	---	---	---	---	---
02/28/90		---	---	---	---	---	---	0.09	---	---	---	---	---	---	---	---
05/14/90		---	---	---	0.63	0.27	---	0.09	---	---	---	---	---	---	---	---
09/06/90		---	---	---	0.66	0.26	---	0.09	---	---	---	---	---	---	---	---
01/04/91		---	---	---	0.69	0.28	---	0.1	---	---	---	---	---	---	---	---
04/15/91		---	---	0.03	---	---	---	---	---	---	---	---	---	---	---	---
04/16/91		---	---	---	0.68	0.29	---	0.12	---	---	---	---	---	---	---	---
10/25/91		---	---	---	0.69	0.29	---	---	---	---	---	---	---	---	---	---
10/28/91		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
10/29/91		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
10/30/91		---	---	---	---	---	---	0.1	---	---	---	---	---	---	---	---
11/14/91		---	---	0.06	---	---	---	---	---	---	---	---	---	---	---	---
11/19/91		---	---	---	---	---	---	0.1	---	---	---	---	---	---	---	---
4/15/92		---	---	0.09	---	---	---	---	---	---	---	---	---	---	---	---
4/22/92		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/11/92		---	---	---	---	---	---	0.1	---	---	---	---	---	---	---	---
09/24/92		---	---	---	---	---	---	0.1	---	---	---	---	---	---	---	---
11/24/92		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/25/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
1/26/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
2/25/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---

Table C-3
April 1986 through November 1999 Inclinometer Secondary Creep Zone Displacement Data
Inclinometers I-11, I-14 through I-27
Penitencia Water Treatment Plant

Date	Inclinometer No	I-11	I-14	I-15	I-16	I-16	I-17	I-19	I-20	I-21	I-22	I-23	I-24	I-25	I-26	I-27
	Depth in feet	84-93	48-51	28-33	32-35.5	25	(6 zones*)	160-164	55-57	75	75	97	40	106-109	28	
		Displacement in inches														
4/23/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/27/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/12/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
12/9/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
12/13/93		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
2/24/94		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
3/1/94		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/27/94		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/18/94		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/1/94		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
2/15/95		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/18/95		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/26/95		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/13/95		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/14/95		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/10/96		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
4/16/96		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
04/18/96		---	---	0.32	---	---	---	---	---	---	---	---	---	---	---	---
04/24/96		---	0.13	---	---	---	---	0.22	---	---	---	---	---	---	---	---
8/23/96		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
8/26/96		---	---	---	---	---	---	---	---	1.86	---		---	---	---	---

Table C-3
April 1986 through November 1999 Inclinometer Secondary Creep Zone Displacement Data
Inclinometers I-11, I-14 through I-27
Penitencia Water Treatment Plant

Date	Inclinometer No	I-11	I-14	I-15	I-16	I-16	I-17	I-19	I-20	I-21	I-22	I-23	I-24	I-25	I-26	I-27
	Depth in feet	84-93	48-51	28-33	32-35.5	25	(6 zones*)	160-164	55-57	75	75	97	40	106-109	28	
		Displacement in inches														
9/3/96		---	---	---	---	---	---	---	---	1.86	---	0.05	---	---	---	---
11/7/96		---	---	---	---	---	---	---	---	---	---	0.06	---	---	---	---
11/12/96		---	---	---	---	---	---	---	---	1.86	---	---	---	---	---	---
6/12/97		---	---	---	---	---	---	---	---	1.9	0.08	0.1	0.08	---	---	---
06/13/97		---	---	---	---	---	0.077	---	---	---	---	---	---	---	---	---
9/15/97		---	---	---	---	---	---	---	---	1.9	0.085	0.12	0.08	---	---	---
09/17/97		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
11/25/97		---	---	---	---	---	---	---	---	---	---	0.12	0.09	---	---	---
12/2/97		---	---	---	---	---	0.093	---	---	---	0.09	---	---	---	---	---
3/2/98		---	---	---	---	---	---	---	---	---	0.14	0.17	---	---	0.065	---
06/22/98		---	---	---	---	---	0.187	---	---	---	---	---	---	---	---	---
6/26/98		---	---	---	---	---	---	---	---	---	---	0.17	0.11	---	---	---
7/13/98		---	---	---	---	---	---	---	---	1.9	0.15	---	---	---	0.067	---
9/28/98		---	---	---	---	---	---	---	---	1.9	0.156	0.18	0.12	---	0.07	---
12/17/98		---	---	---	---	---	---	---	---	1.9	---	0.19	---	---	0.07	---
12/19/98		---	---	---	---	---	---	---	---	---	0.156	---	0.11	---	---	---
2/19/99		---	---	---	---	---	---	---	---	---	---	0.21	0.12	---	---	---
2/24/99		---	---	---	---	---	---	---	---	---	---	---	---	---	---	---
3/1/99		---	---	---	---	---	---	---	---	---	0.145	---	---	---	0.07	---
5/27/99		---	---	---	---	---	0.218	---	---	1.9	---	0.21	---	---	0.07	---
6/2/99		---	---	---	---	---	---	---	---	---	0.152	---	0.13	---	---	---

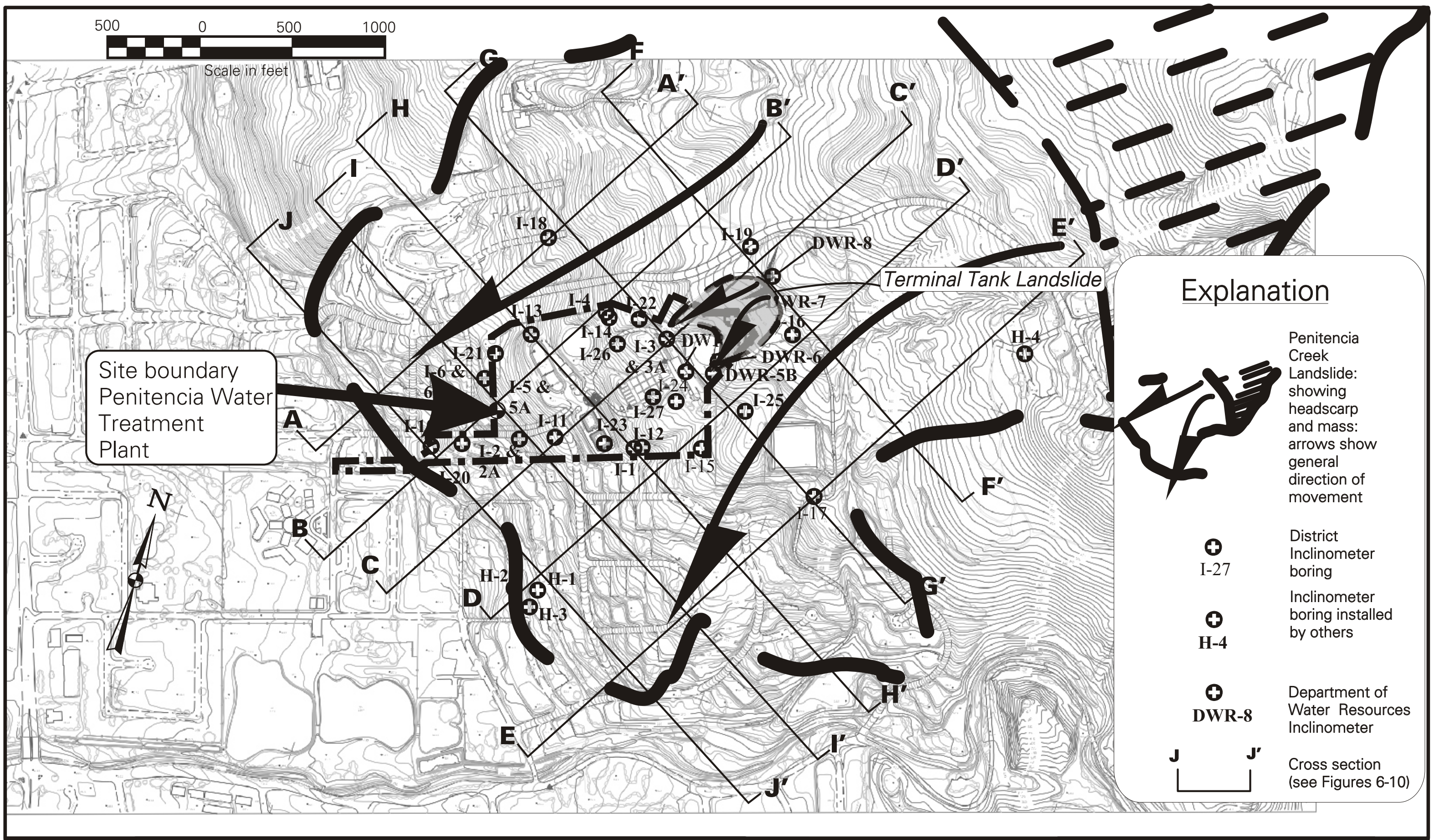
Table C-3

April 1986 through November 1999 Inclinometer Secondary Creep Zone Displacement Data

Inclinometers I-11, I-14 through I-27

Penitencia Water Treatment Plant

Date	Inclinometer No	I-11	I-14	I-15	I-16	I-16	I-17	I-19	I-20	I-21	I-22	I-23	I-24	I-25	I-26	I-27
	Depth in feet	84-93	48-51	28-33	32-35.5	25	(6 zones*)	160-164	55-57	75	75	97	40	106-109	28	
		Displacement in inches														
8/24/99		---	---	---	---	---	---	---	---	---	---	0.24	---	---	---	---
8/25/99		---	---	---	---	---	---	---	---	---	---	---	0.14	---	0.08	---
8/26/99		---	---	---	---	---	---	---	---	1.87	0.16	---	---	---	---	---
11/13/99		---	---	---	---	---	---	---	---	1.88	---	0.24	0.13	---	0.08	0.1
11/15/99		---	---	---	---	---	0.22	---	---	---	0.15	---	---	---	---	---


500
0
500
1000
Scale in feet
G
F
A′
H
B′
C′
I
D′
J
E′
I-18
I-19
DWR-8
Terminal Tank Landslide
I-4
I-22
DWR-7
I-13
-16
I-14
I-3
DW
I-21
I-26
& 3A
DWR-6
I-6 &
6
I-24
DWR-5B
H-4
I-5 &
5A
I-27
I-25
Site boundary Penitencia Water Treatment Plant
A
I-11
I-23
I-12
I-1
I-2 &
2A
I-1
I-15
I-20
F′
N
I-17
B
C
G′
H-2
H-1
D
H-3
H′
E
I′
J′
Explanation
Penitencia Creek Landslide: showing headscarp and mass: arrows show general direction of movement
I-27
District Inclinometer boring
H-4
Inclinometer boring installed by others
DWR-8
Department of Water Resources Inclinometer
J
J′
Cross section (see Figures 6-10)

Notes: 1) Base map by Transvision, dated 2/25/00, elevations based on NAVD 1988 datum, Photography Air Flight Service, flown 12/99, California Coordinate System Zone 3
2) File: W:\geology\penitencia creek landslide\figures\figc-1a.cdr

DATE 9/28/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

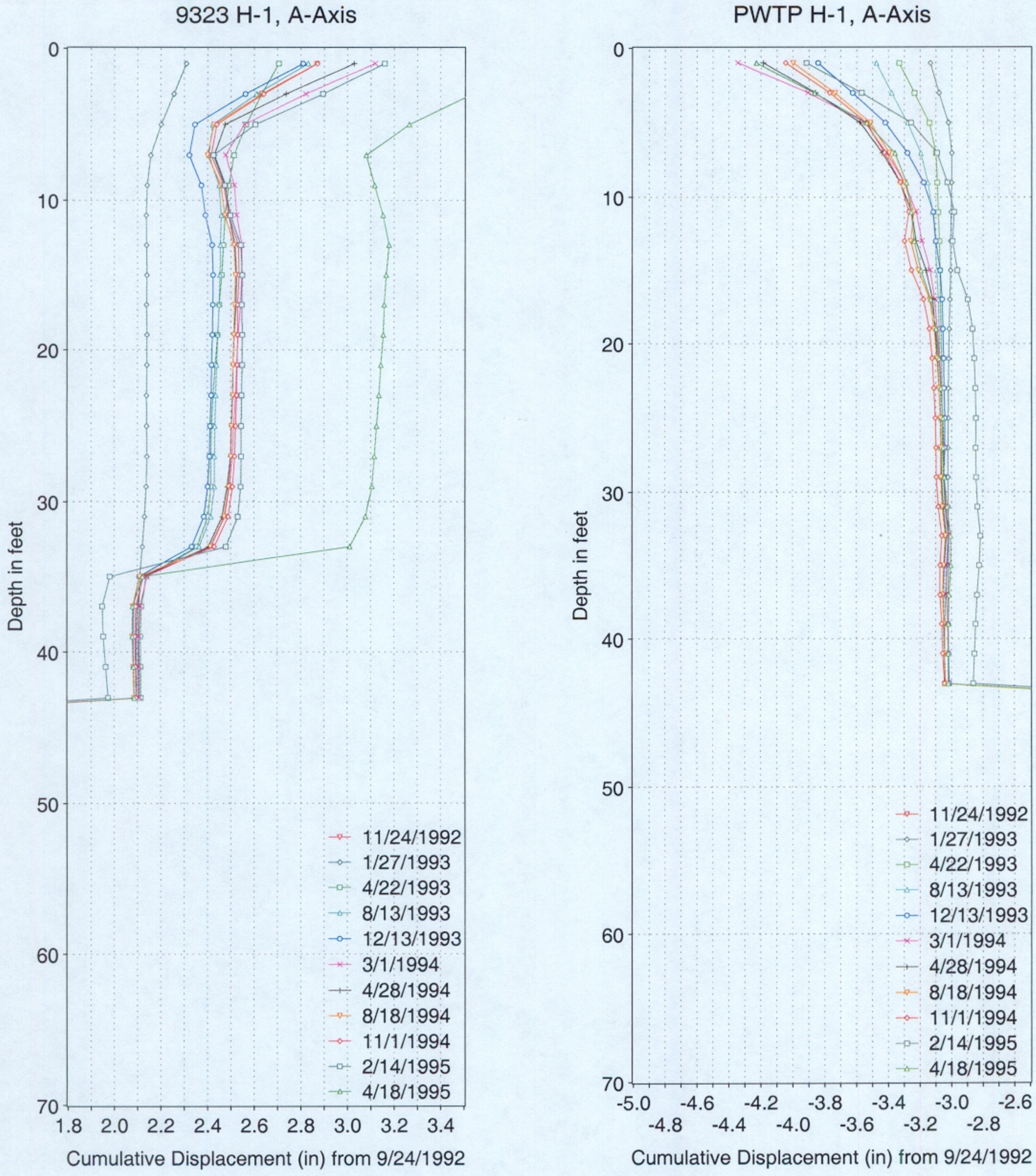
Inclinometer Boring Locations

Penitentia Creek Landslide and Vicinity

Penitencia Water Treatment Plant

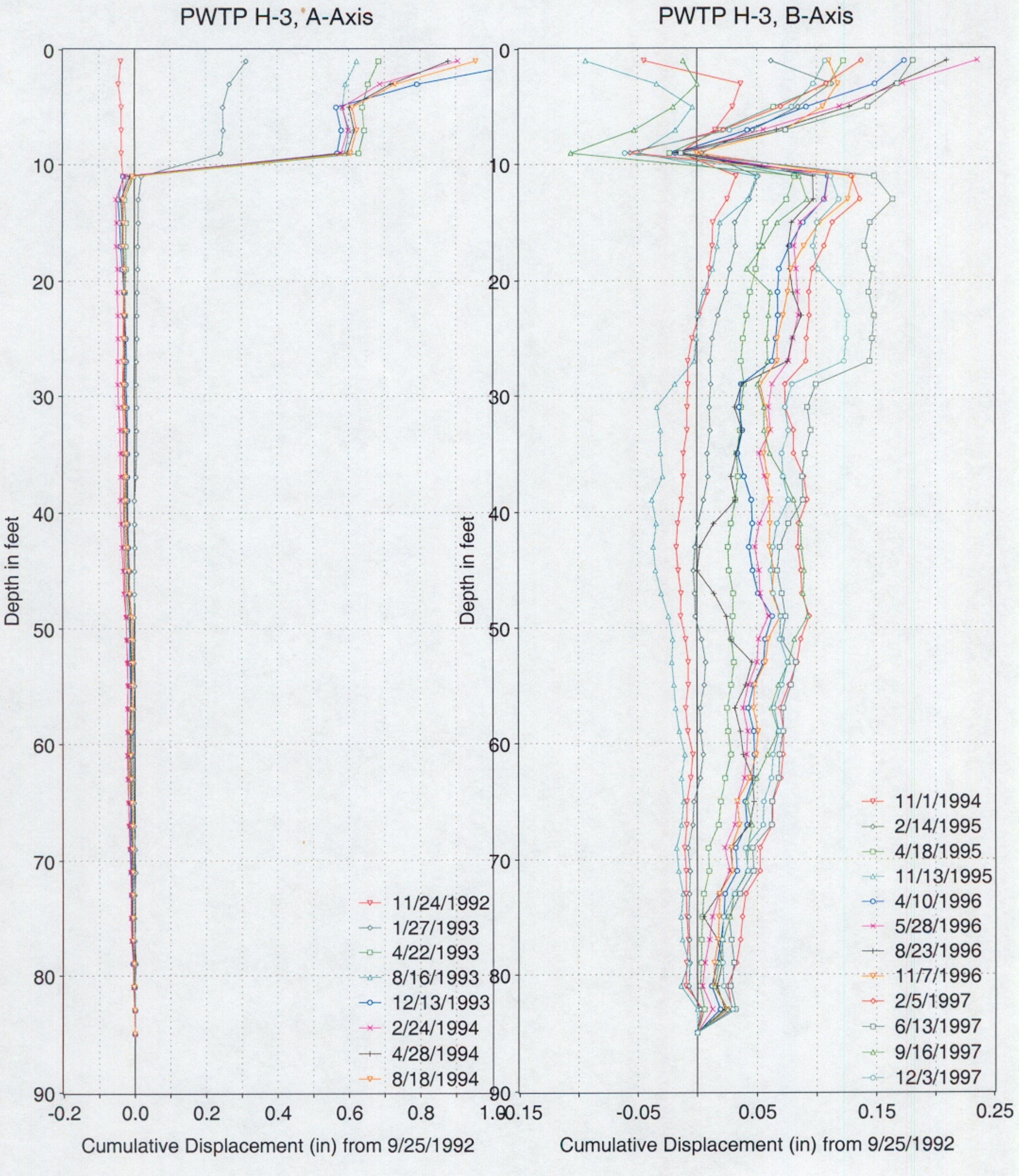
SCALE 1"= 500′

FIGURE C-1



Santa Clara Valley Water District
Penitencia Water Treatment Plant
Landslide Monitoring - rotated 35 degrees counterclockwise
Inclinometer Report

Figure C-2



PWTP H-3, A-Axis

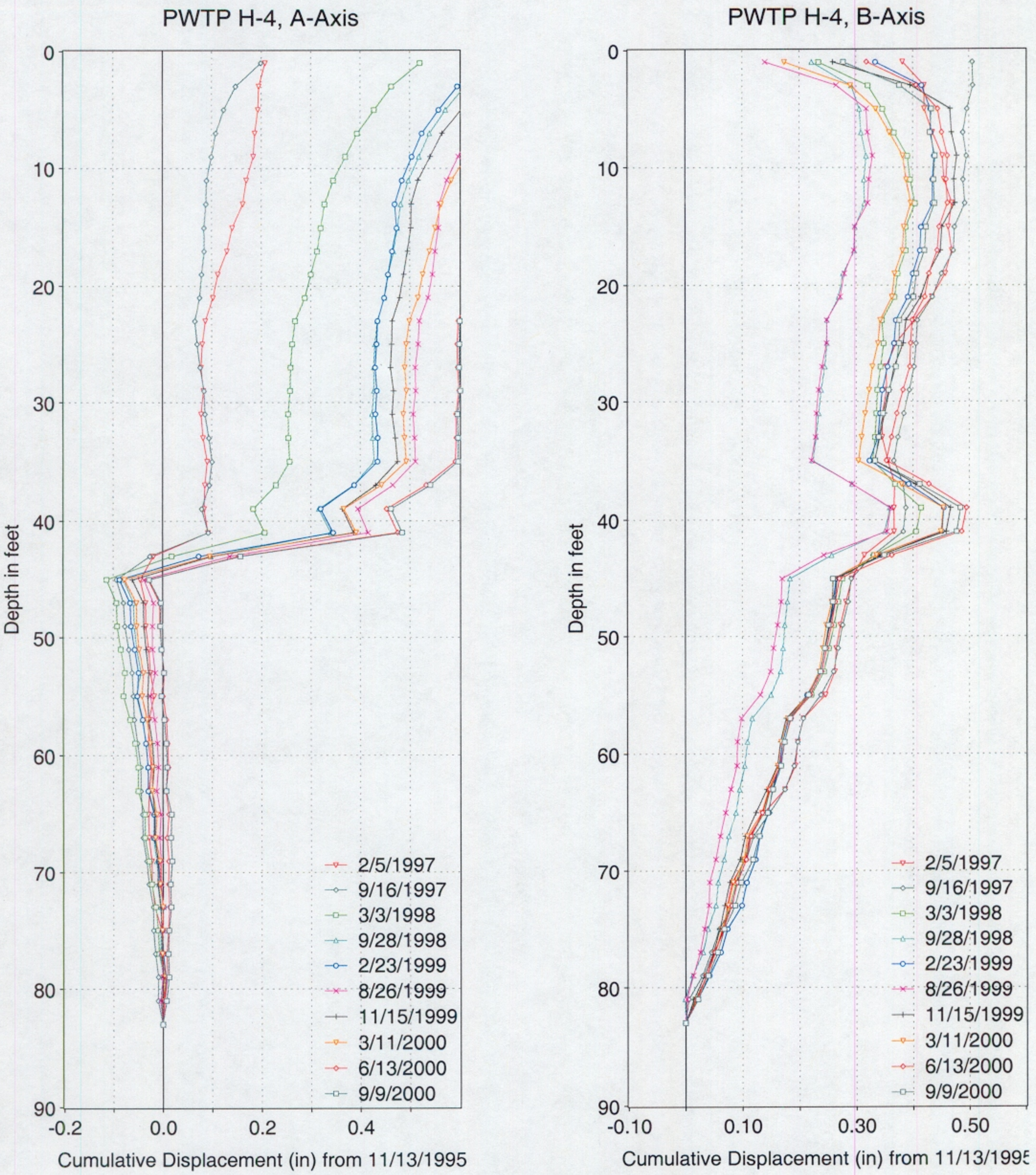
PWTP H-3, B-Axis

Santa Clara Valley Water District
Penitencia Water Treatment Plant

Landslide Monitoring - Zero Deg Rotate A&B+

Inclinometer Report

Figure C-3

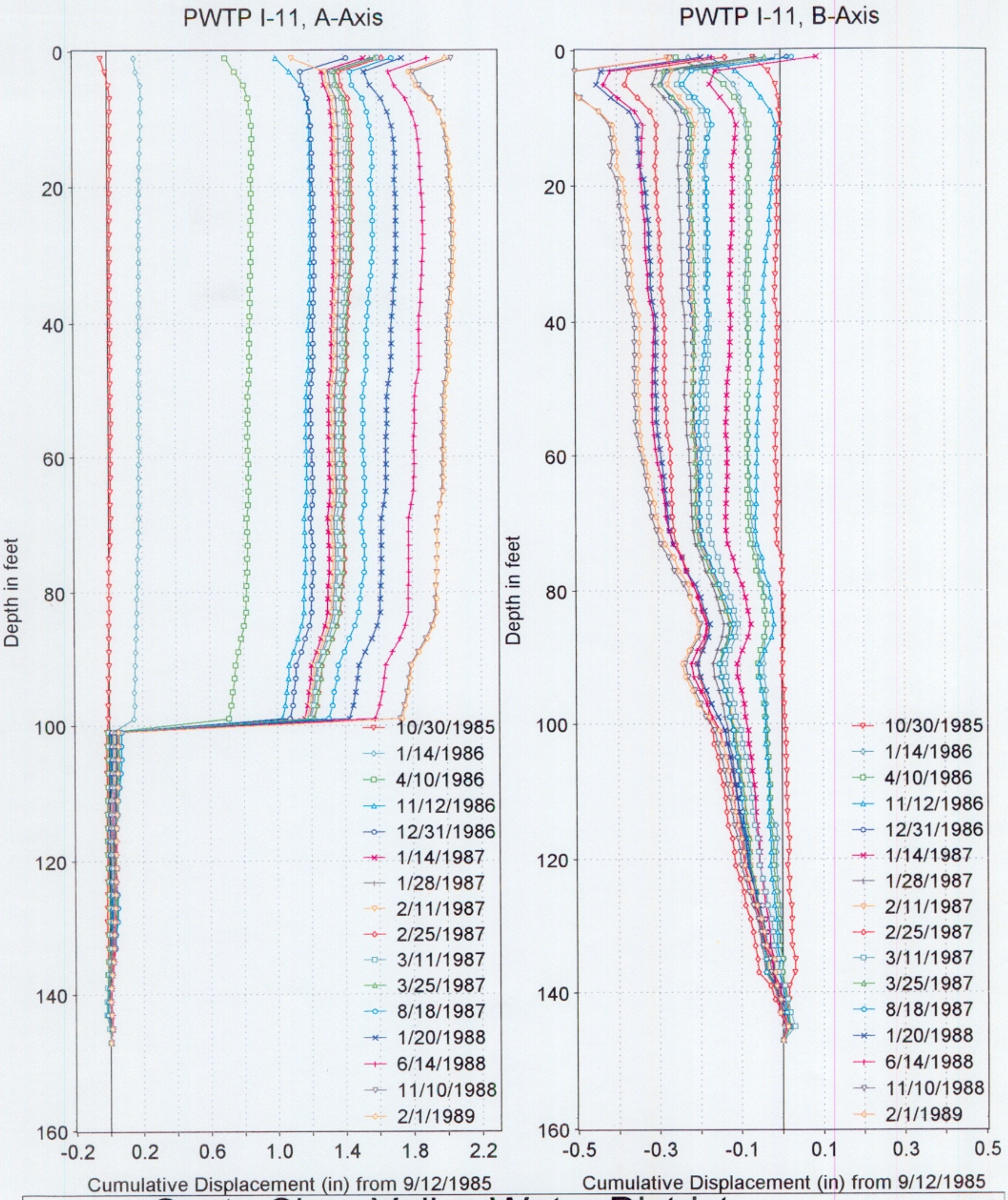


Santa Clara Valley Water District
Penitentia Water Treatment Plant

Landslide Monitoring - rotated 30 degrees clockwise A&B+

Inclinometer Report

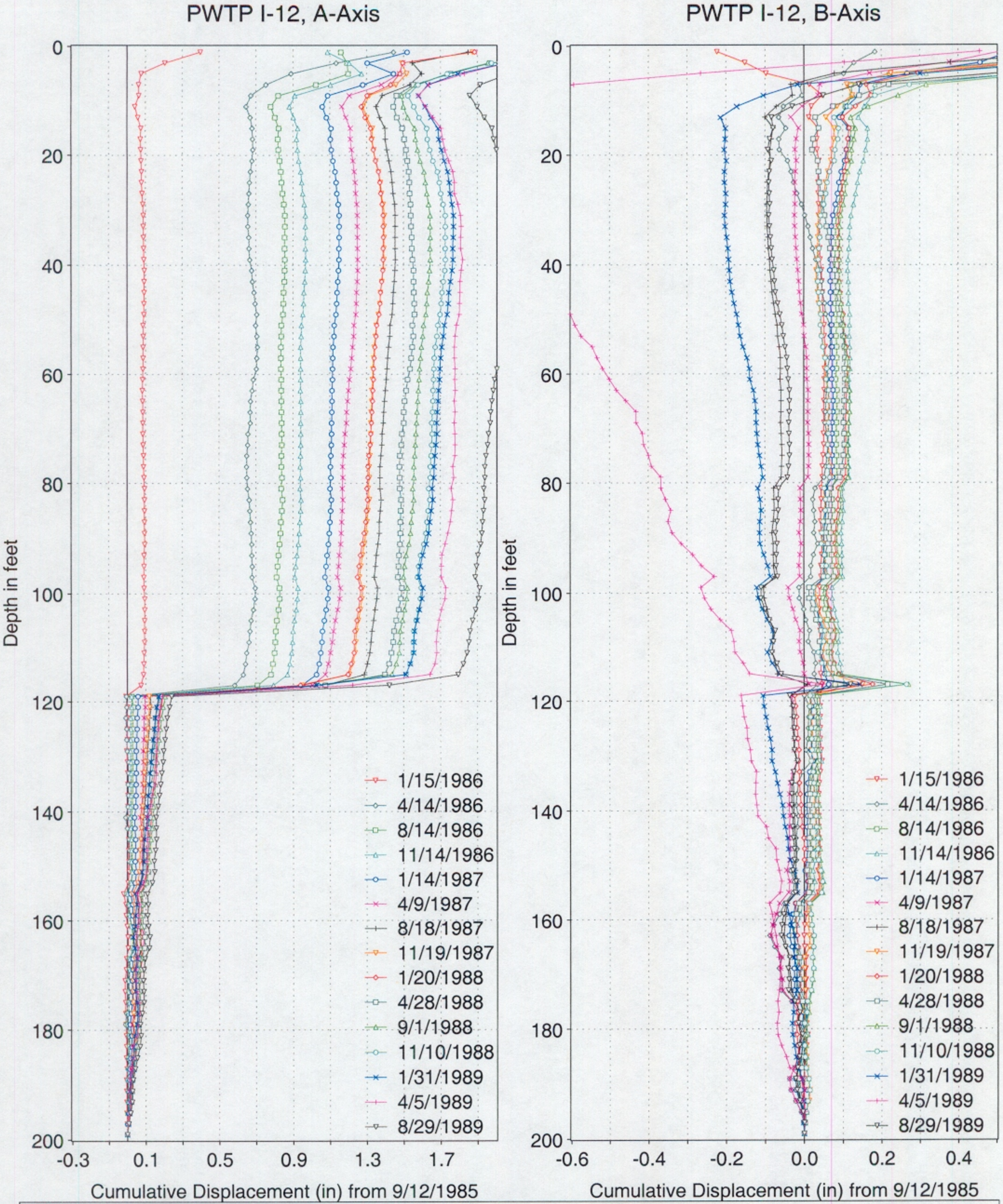
Figure C-4



Santa Clara Valley Water District
Penitencia Water Treatment Plant

Landslide Monitoring - rotated 30 degrees counterclockwise
Inclinometer report of I-11

Figure C-5



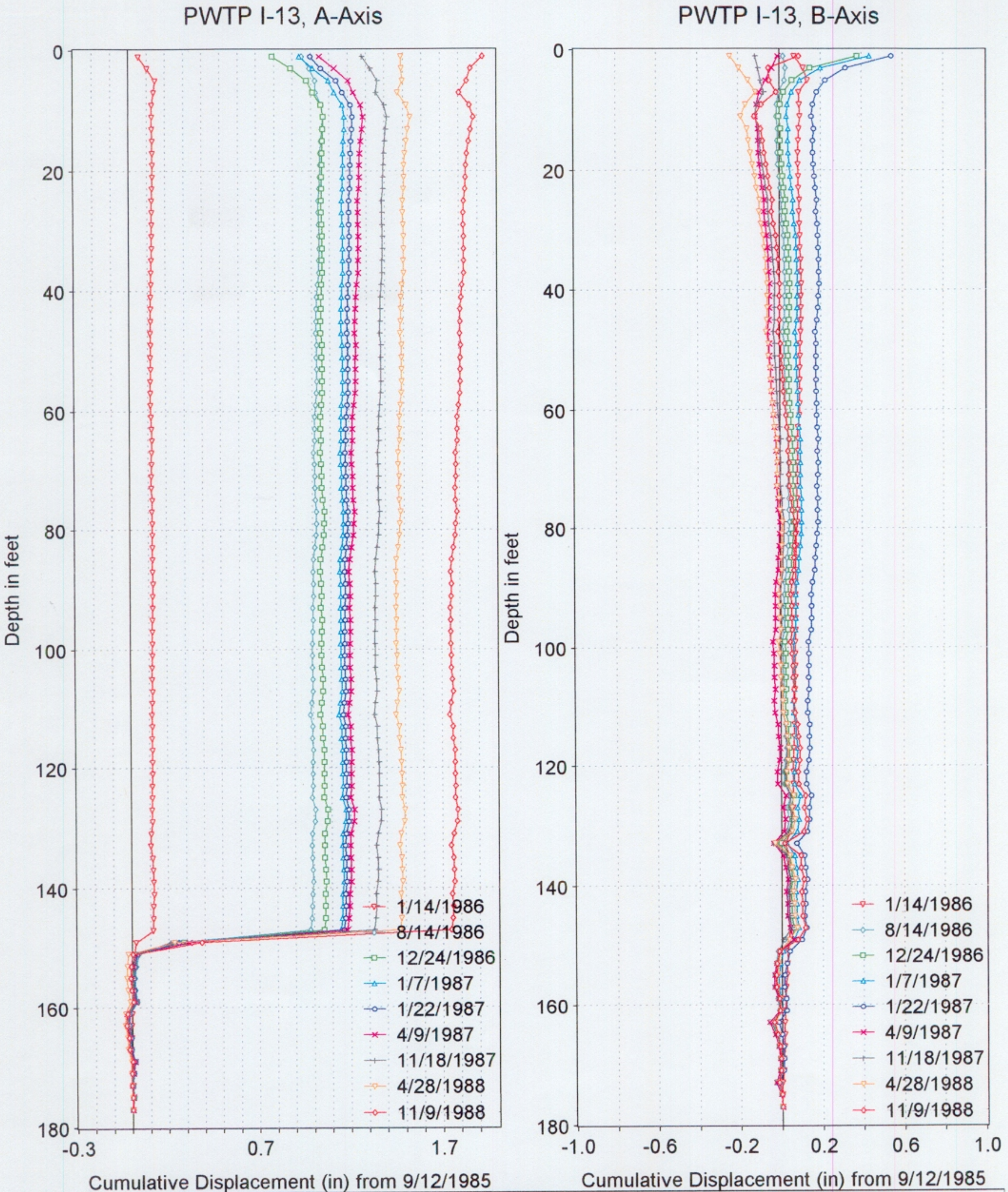
Santa Clara Valley Water District

Penitencia Water Treatment Plant

Landslide Monitoring - rotated 65 degrees counter clockwise

Inclinometer report of I-12

Figure C-6

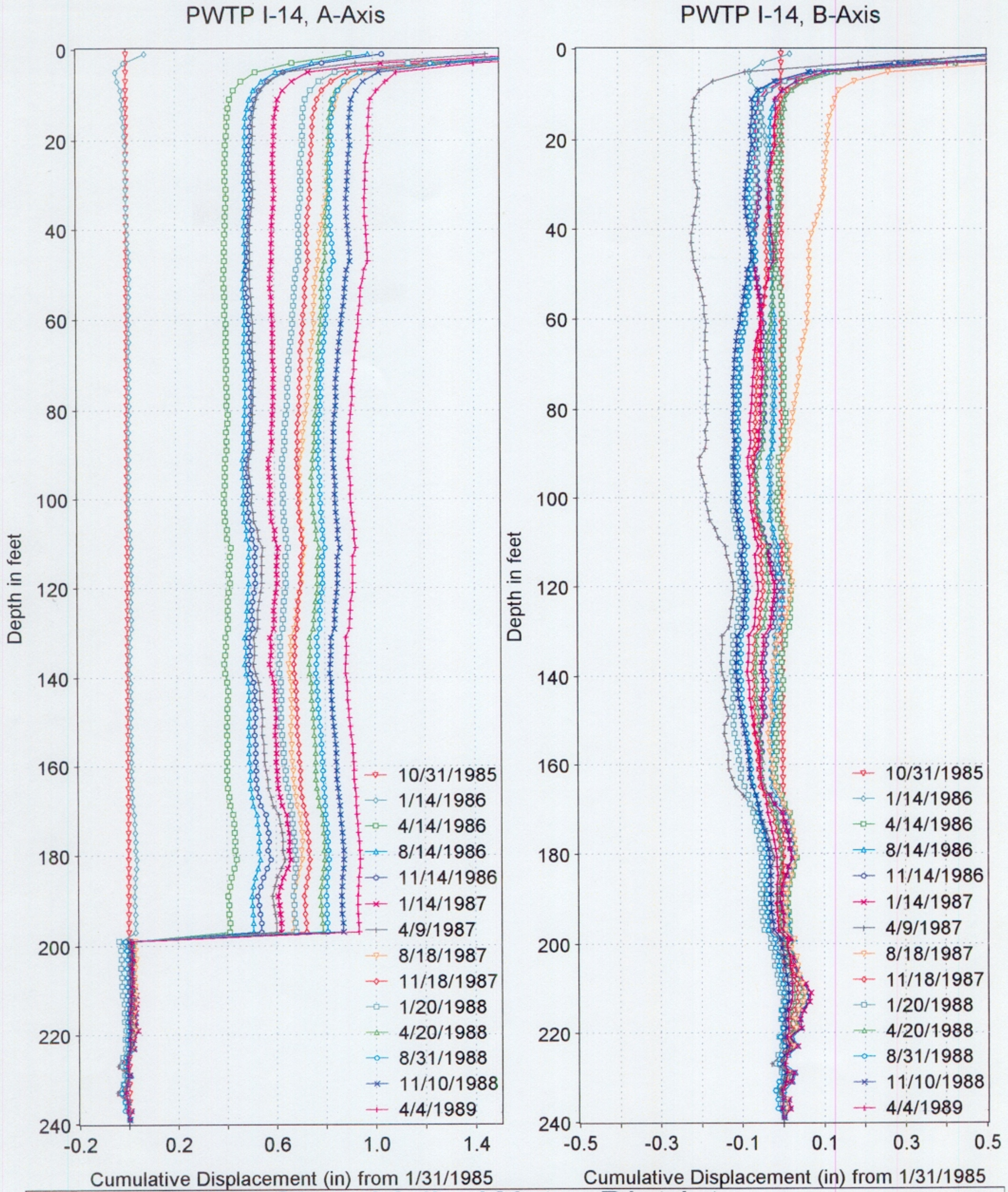


Santa Clara Valley Water District

Penitencia Wate Treatment Plant

Landslide Monitoring - rotated 40 degrees counter clockwise
Inclinometer Report of I-13

Figure C-7



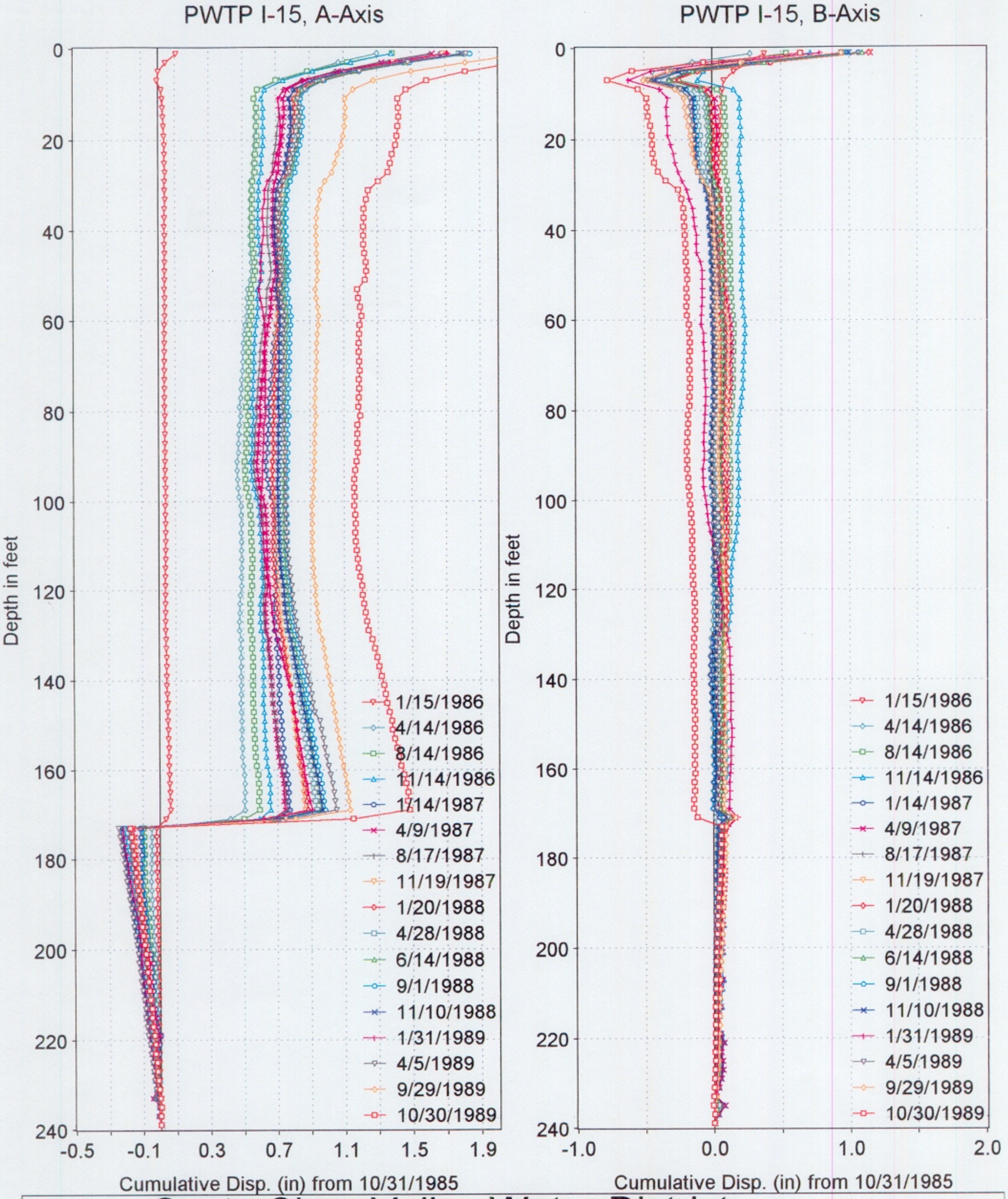
Santa Clara ValleyWater District

Penitencia Water Treatment Plant

Landslide Monitoring - rotated 4 degrees clockwise

Inclinometer Report of I-14

Figure C-8



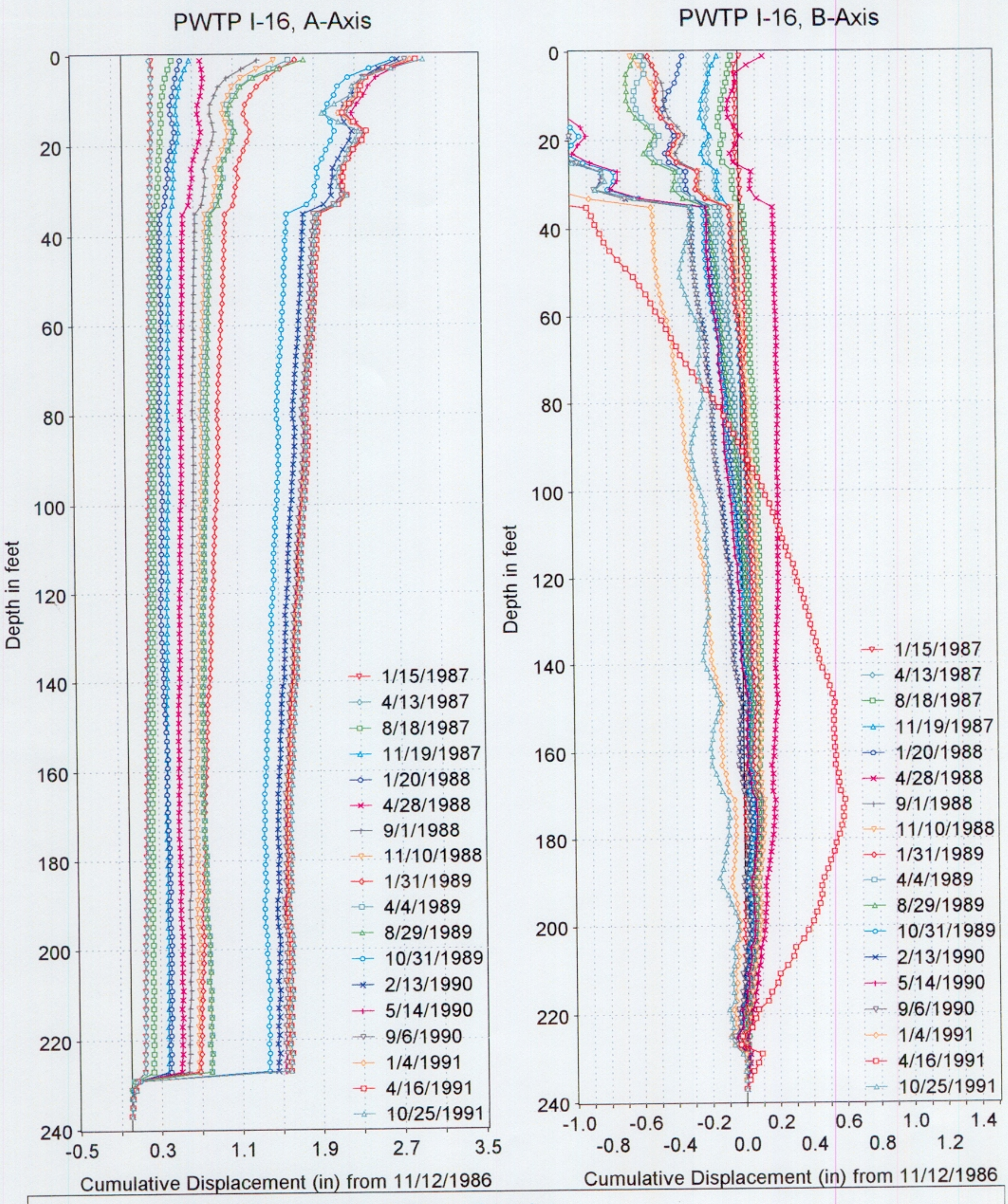
Santa Clara Valley Water District

Penitencia Water Treatment Plant

Landslide Monitoring - rotated 50 degrees counterclockwise

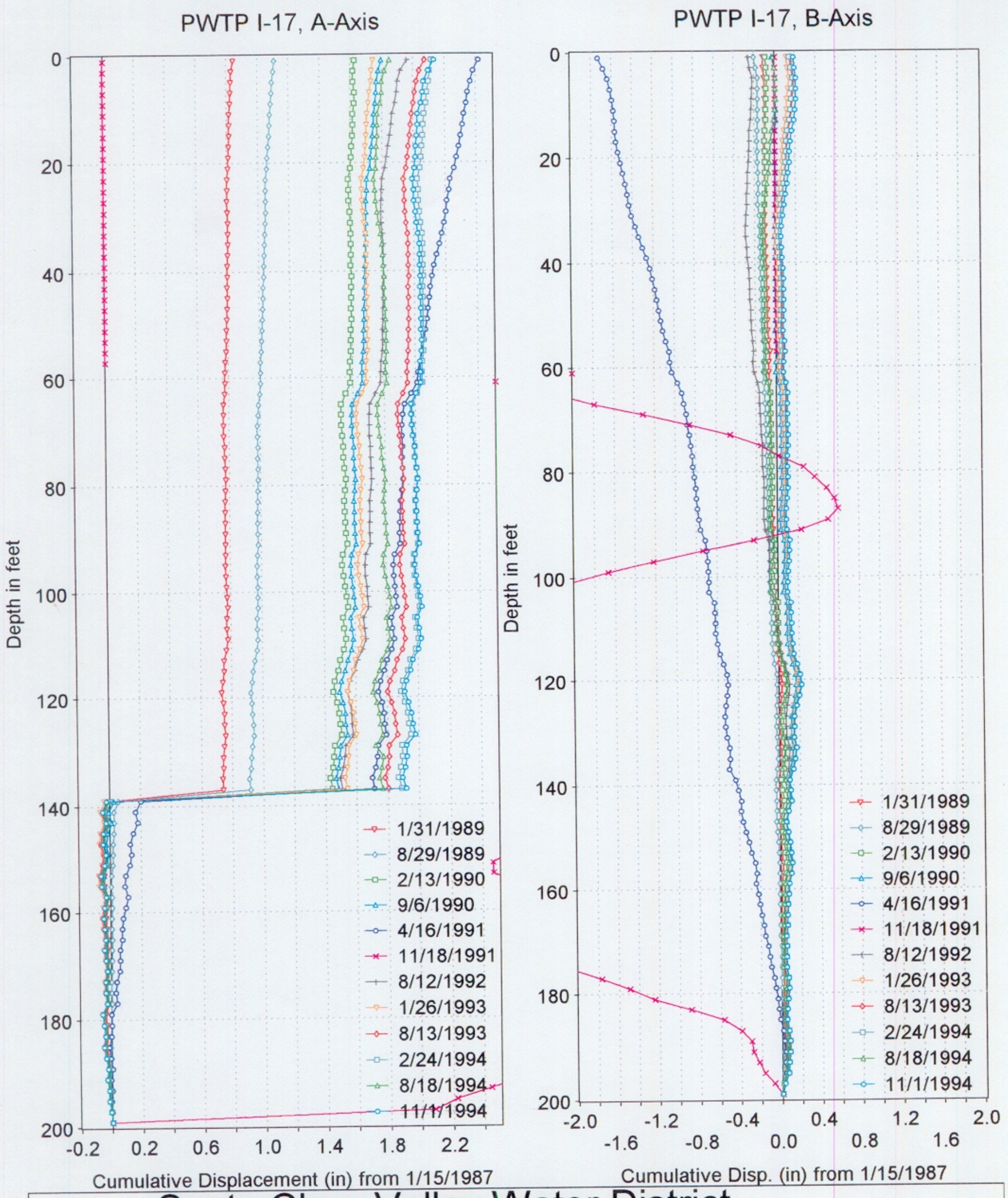
Inclinometer Report of I-15

Figure C-9



Santa Clara Valley Water District
Penitencia Water Treatment Plant
Landslide Monitoring - not rotated
Inclinometer Report of I-16

Figure C-10



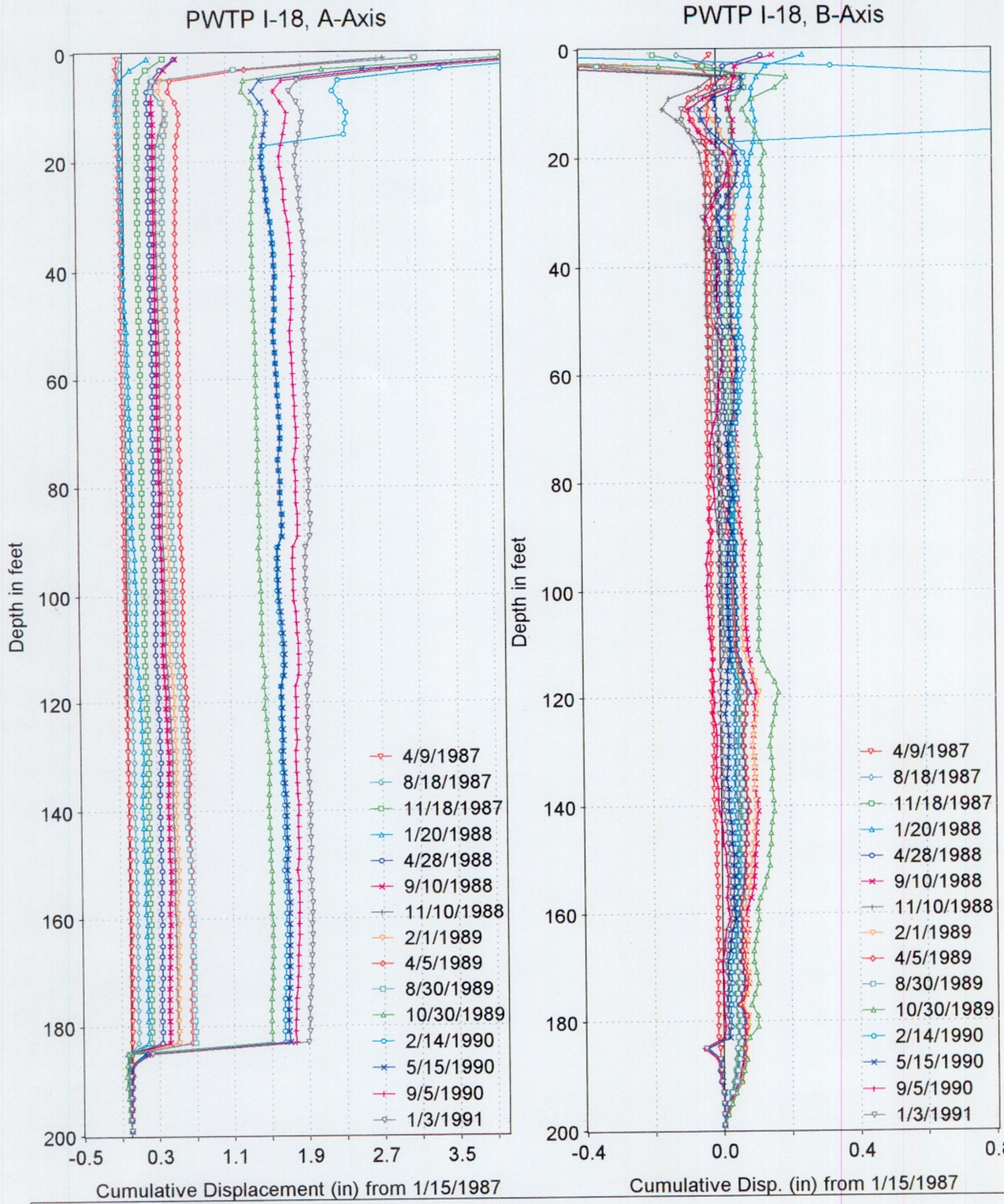
Santa Clara Valley Water District

Penitencia Wate Treatment Plant

Landslide Monitoring - rotated 25 degrees counterclockwise A&B+

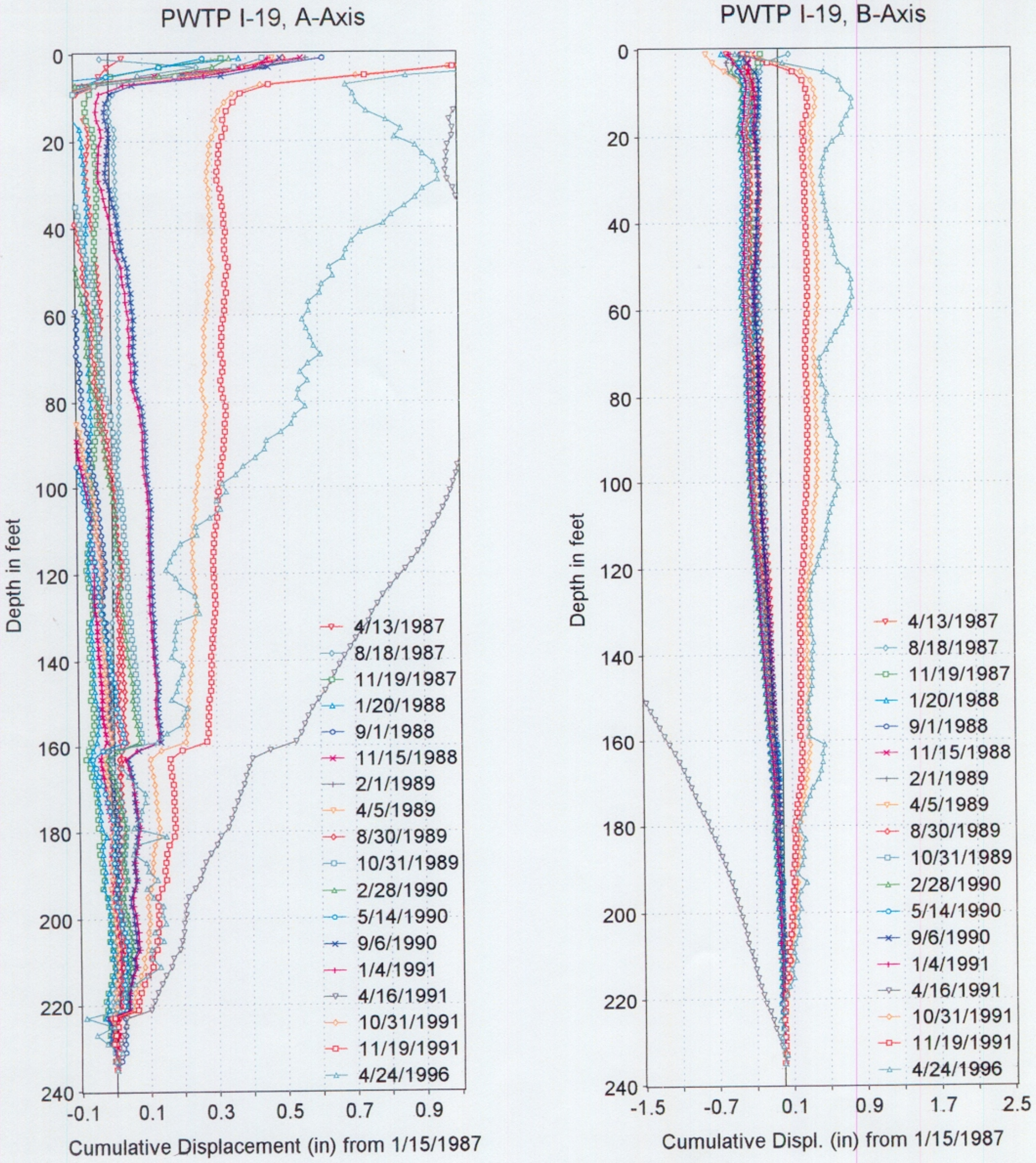
Inclinometer Report of I-17

Figure C-11



Santa Clara Valley Water District
Penitencia Water Treatment Plant
Landslide Monitoring - rotated 45 degrees clockwise
Inclinometer Report of I-18

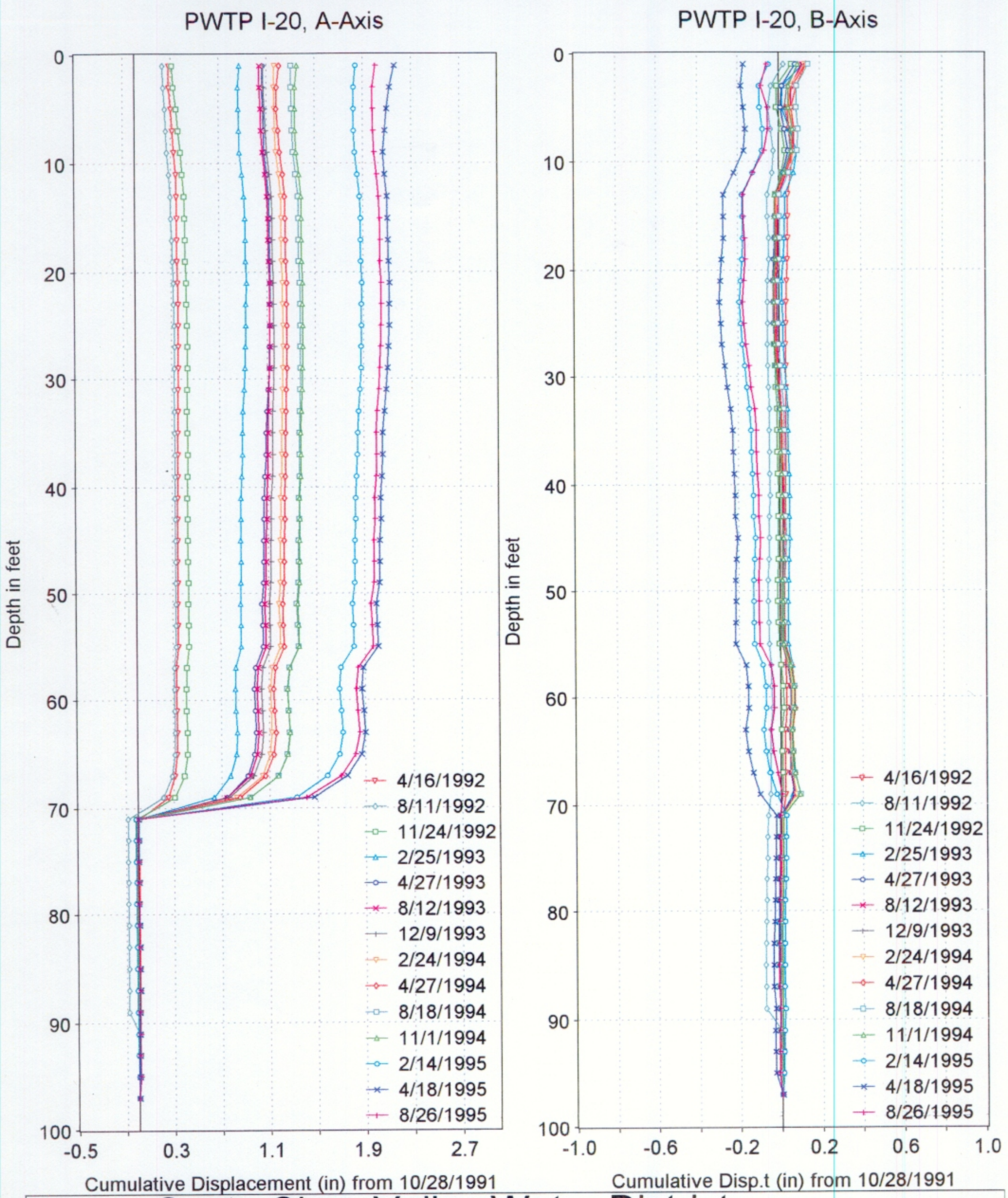
Figure C-12



Santa Clara Valley Water District
Penitencia Water Treatment Plant

Landslide Monitoring - rotated 20 degrees counterclockwise
Inclinometer report of I-19

Figure C-13



PWTP I-20, A-Axis

PWTP I-20, B-Axis

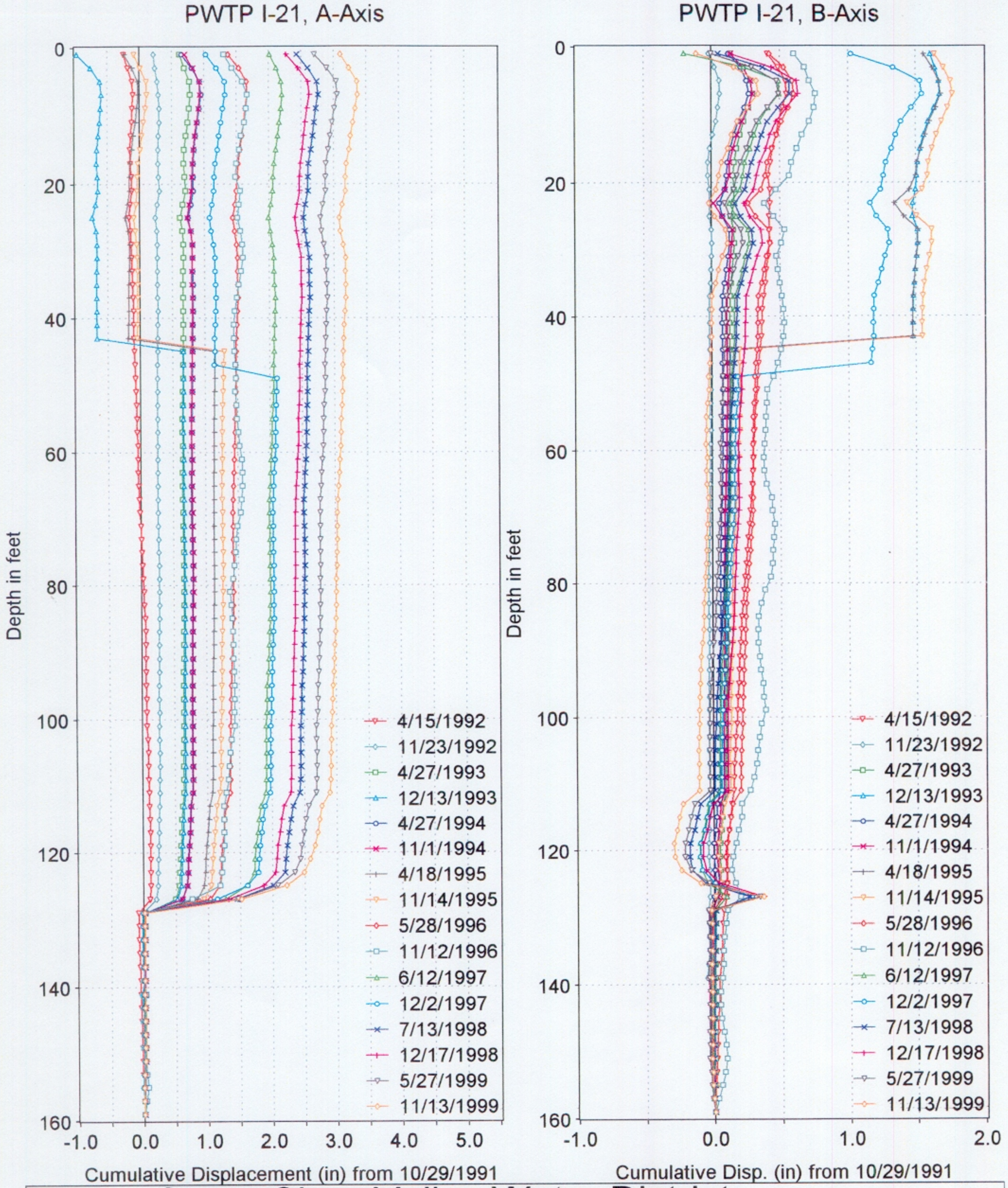
Santa Clara Valley Water District

Penitencia Water Treatment Plant

Landslide Monitoring - rotated 42 degrees counterclockwise

Inclinometer Report of I-20

Figure C-14

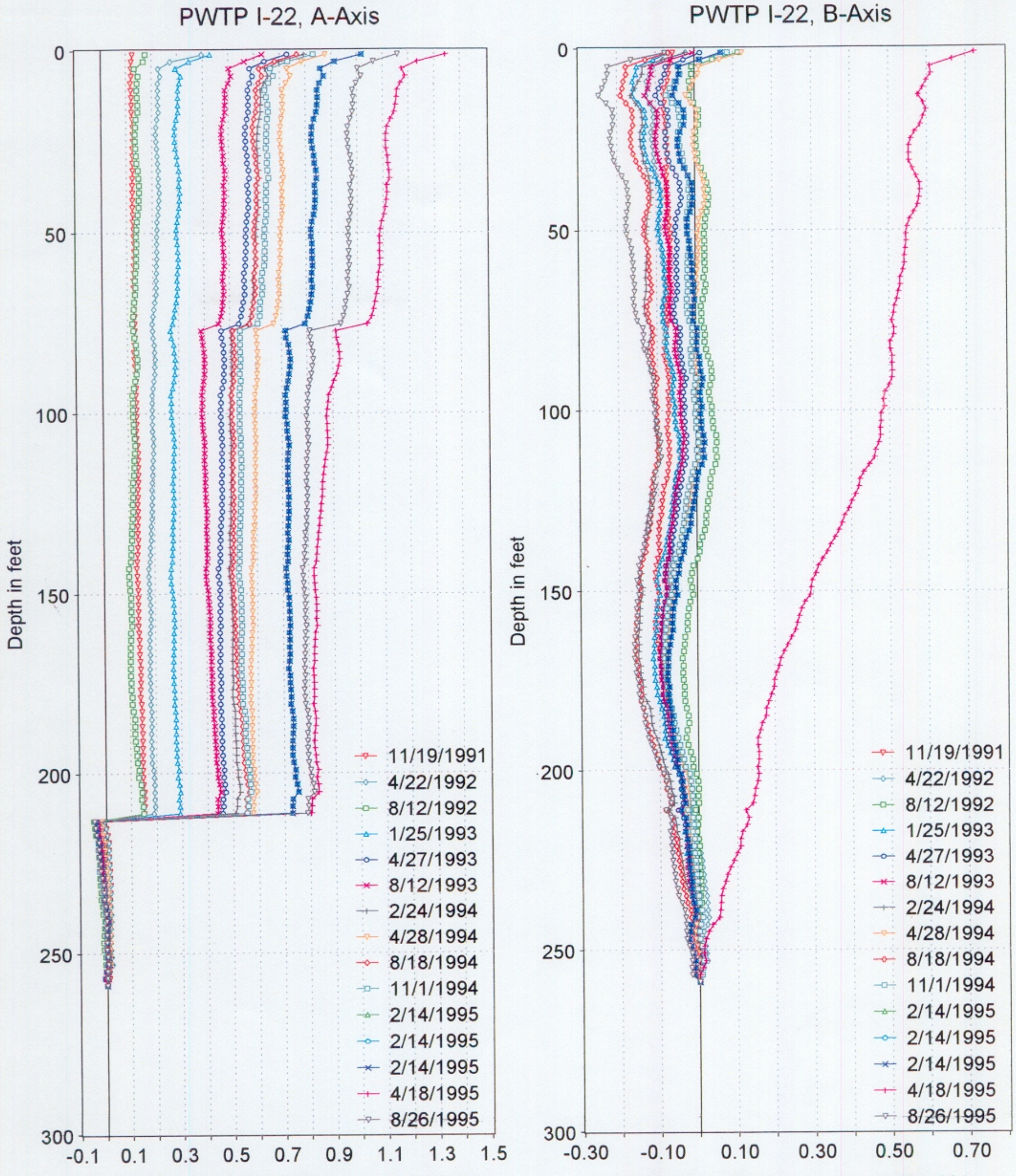


Santa Clara Valley Water District

Penitencia Water Treatment Plant

Landslide Monitoring - rotated 45 degrees counterclockwise
Inclinometer Report of I-21

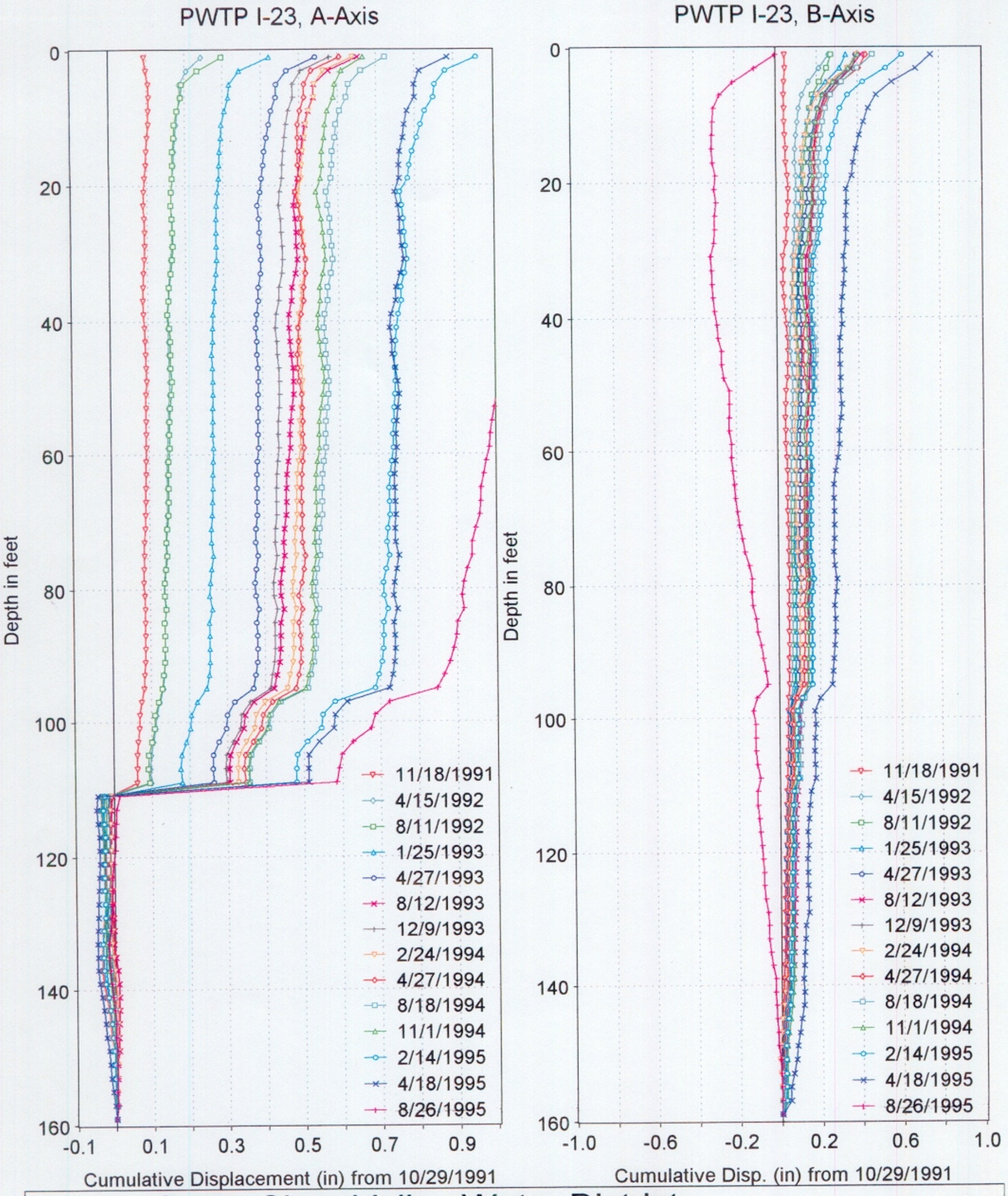
Figure C-15



Santa Clara Valley Water District
Penitencia Water Treatment Plant

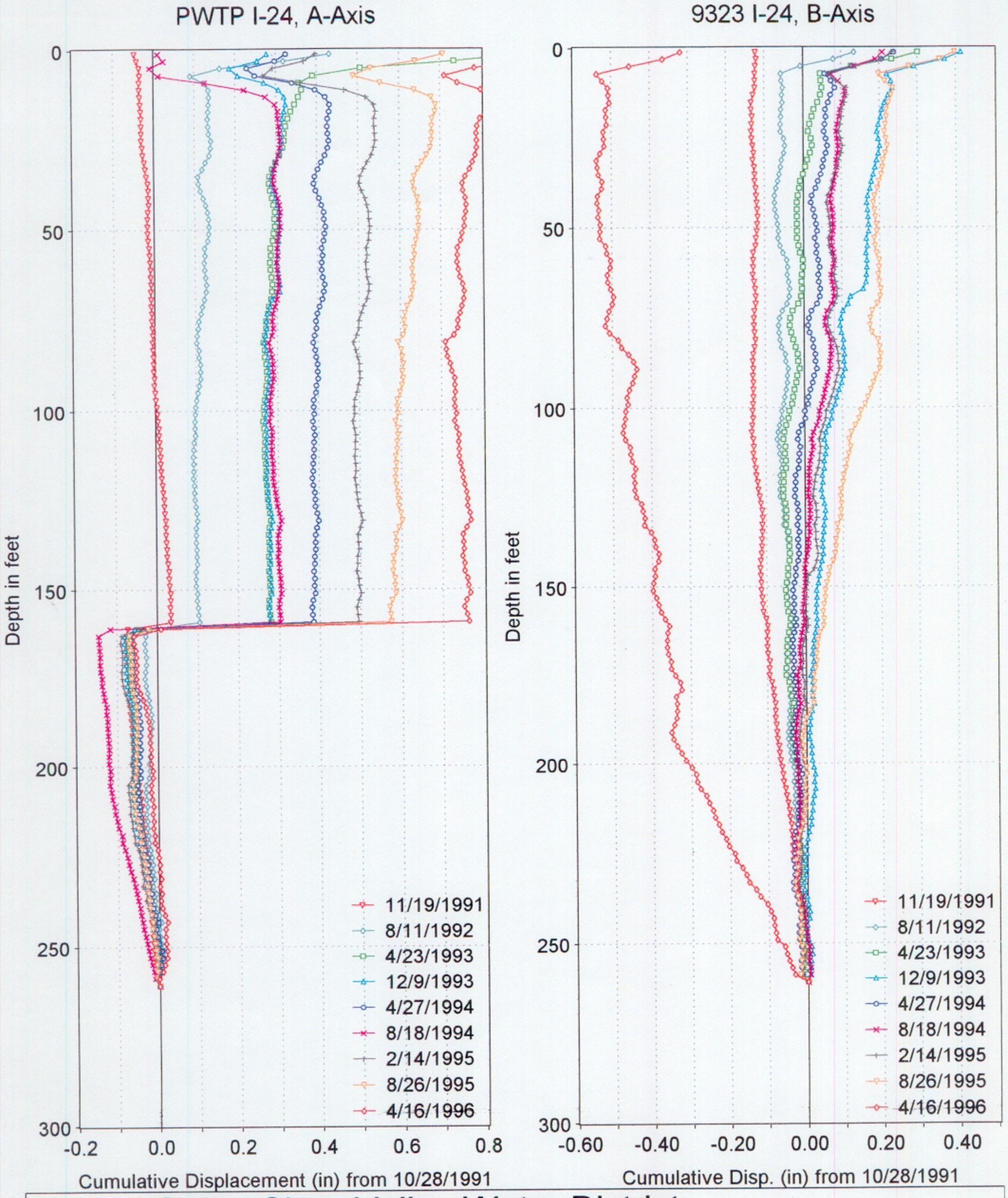
Landslide Monitoring - rotated 20 deg clockwise
Inclinometer Report of I-22

Figure C-16



Santa Clara Valley Water District
Penitencia Water Treatment Plant
Landslide Monitoring - rotated 15 deg counterclockwise
Inclinometer Report of I-23

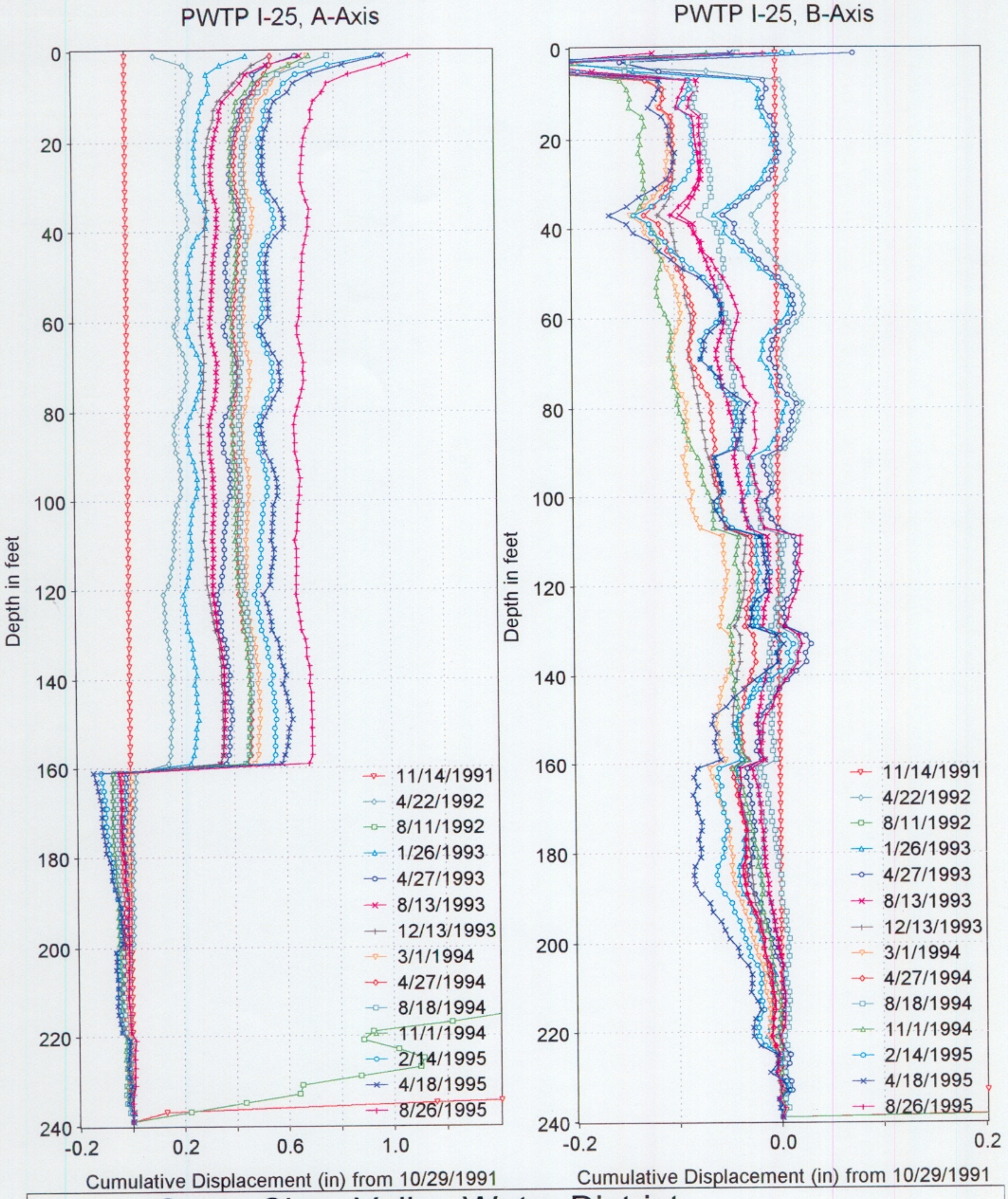
Figure C-17



Santa Clara Valley Water District

Penitencia Water Treatment Plant
Landslide Monitoring - rotated 5 deg counterclockwise
Inclinometer Report of I- 24

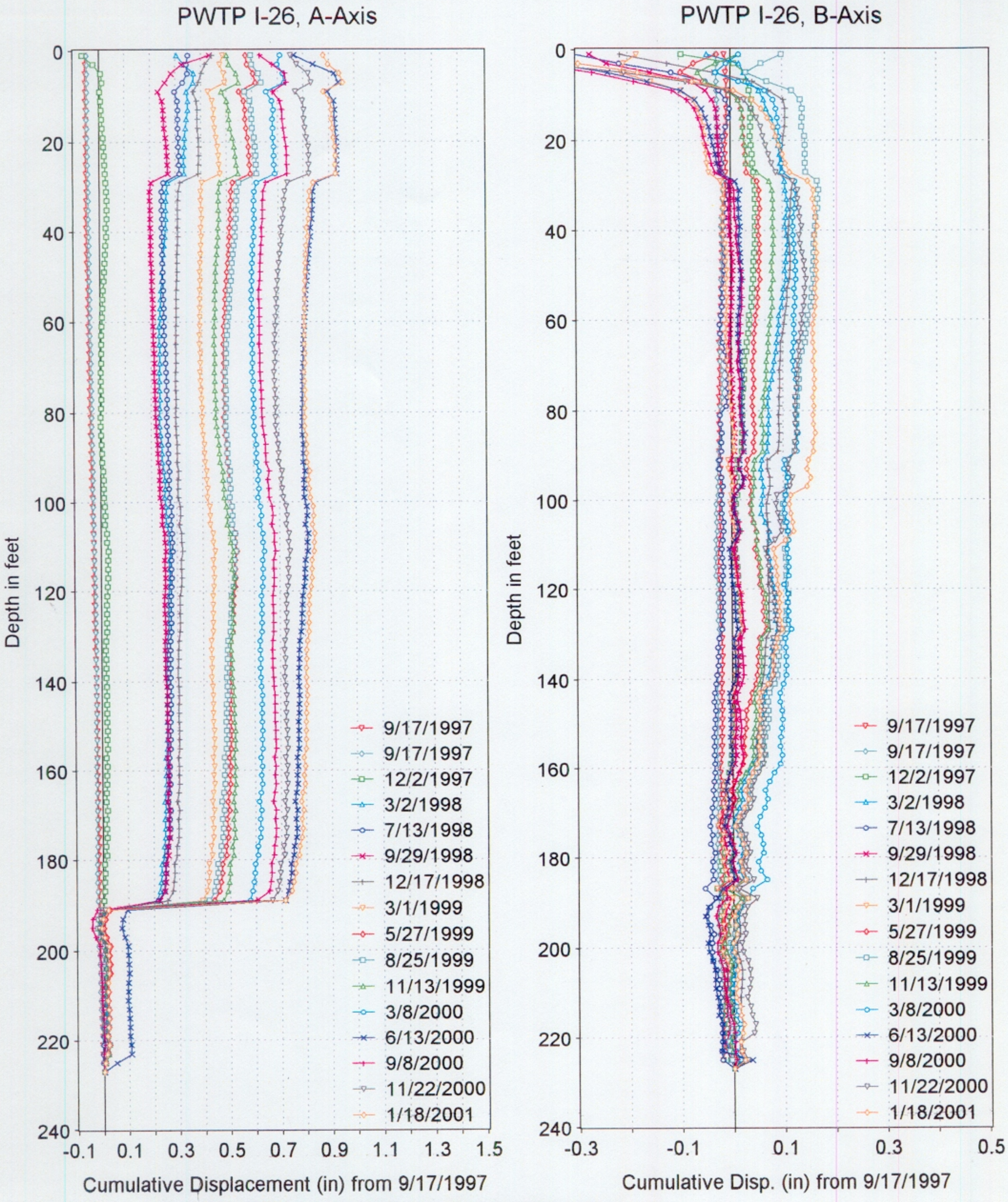
Figure C-18



Santa Clara Valley Water District
Penitencia Water Treatment Plant

Landslide Monitoring -rotated 70 degrees counterclockwise
Inclinometer Report of I-25

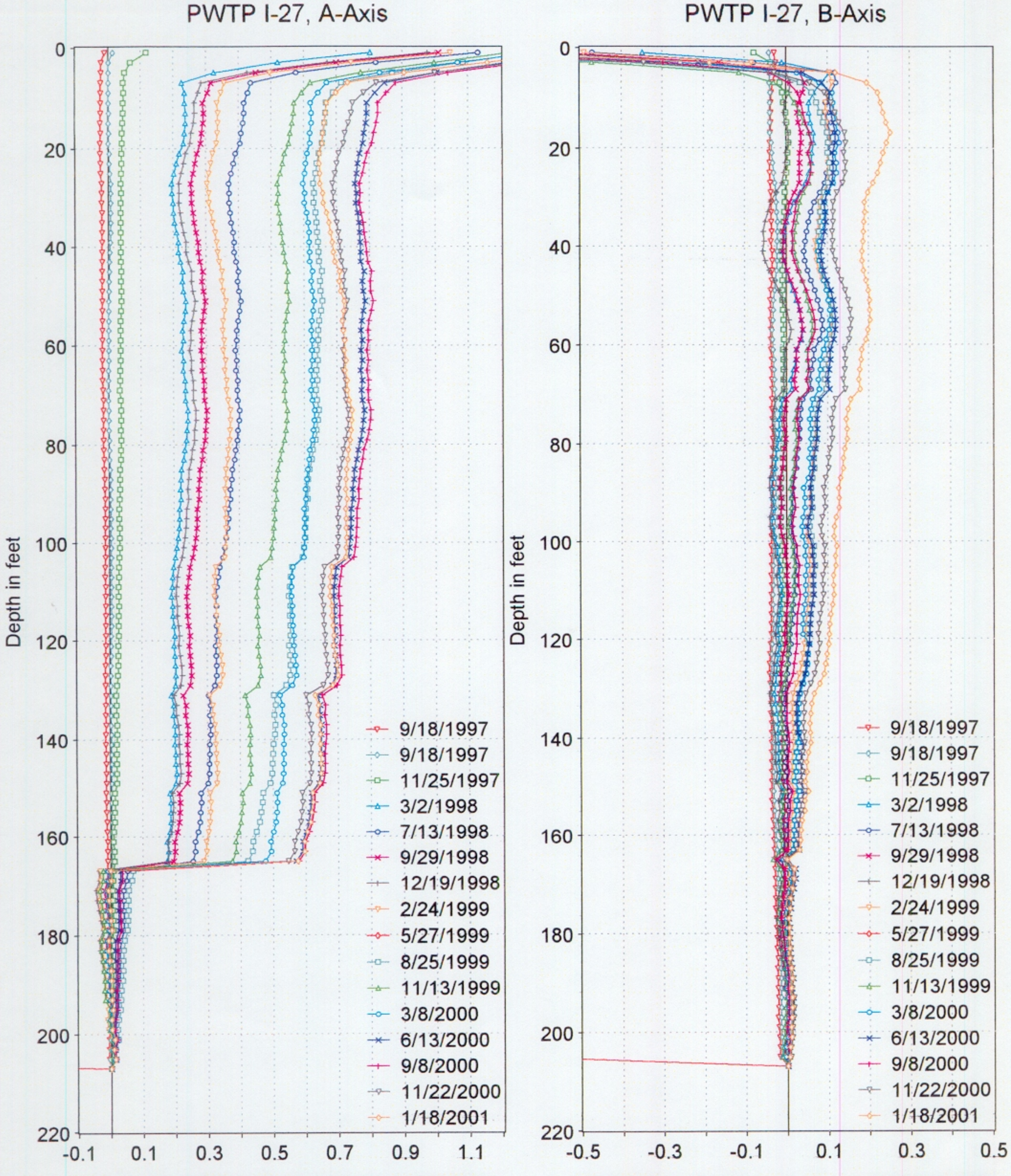
Figure C-19



Santa Clara Valley Water District
Penitencia Water Treatment Plant

Landslide Monitoring - rotated 5 degrees clockwise
Inclinometer Report of I-26

Figure C-20



Santa Clara Valley Water District
Penitencia Water Treatment Plant
Landslide Monitoring-rotated 12 deg clockwise
Inclinometer Report

Figure C-21

APPENDIX D

HISTORICAL AERIAL PHOTOGRAPHS

This appendix presents historical aerial photographs of the Penitencia Creek Landslide collected by the District. Table D-1 lists the photographs covering a span of about 45 years (from 1939 to 1985), including several from each decade at various times of the year, with scales ranging from 1:6,000 to 1:60,000. Each set of photographs under the Photo No. column of Table D-1 consists of at least two photographs that form a stereo pair, which are used to see features in 3 dimensions. Most of the photographs were taken in black and white, while two photographs were taken in color (Photo Nos. 21 & 28, taken in 1974 & 1985, respectively), and one was taken in color infrared (Photo No. 26, 1982).

Historical aerial photographs are important to engineering geologic mapping because 1) they show important physical and geologic features not evident on the ground surface, 2) they form a record of past conditions, 3) they show conditions prior to development, and 4) they provide more detailed information than can be given on a geologic map. Aerial photographs are useful for gaining insight into ground conditions of engineering significance such as soil type, drainage, the presence of marshy areas, unstable ground, spring lines, rock outcrops, and linear features that could denote faulting or landsliding.

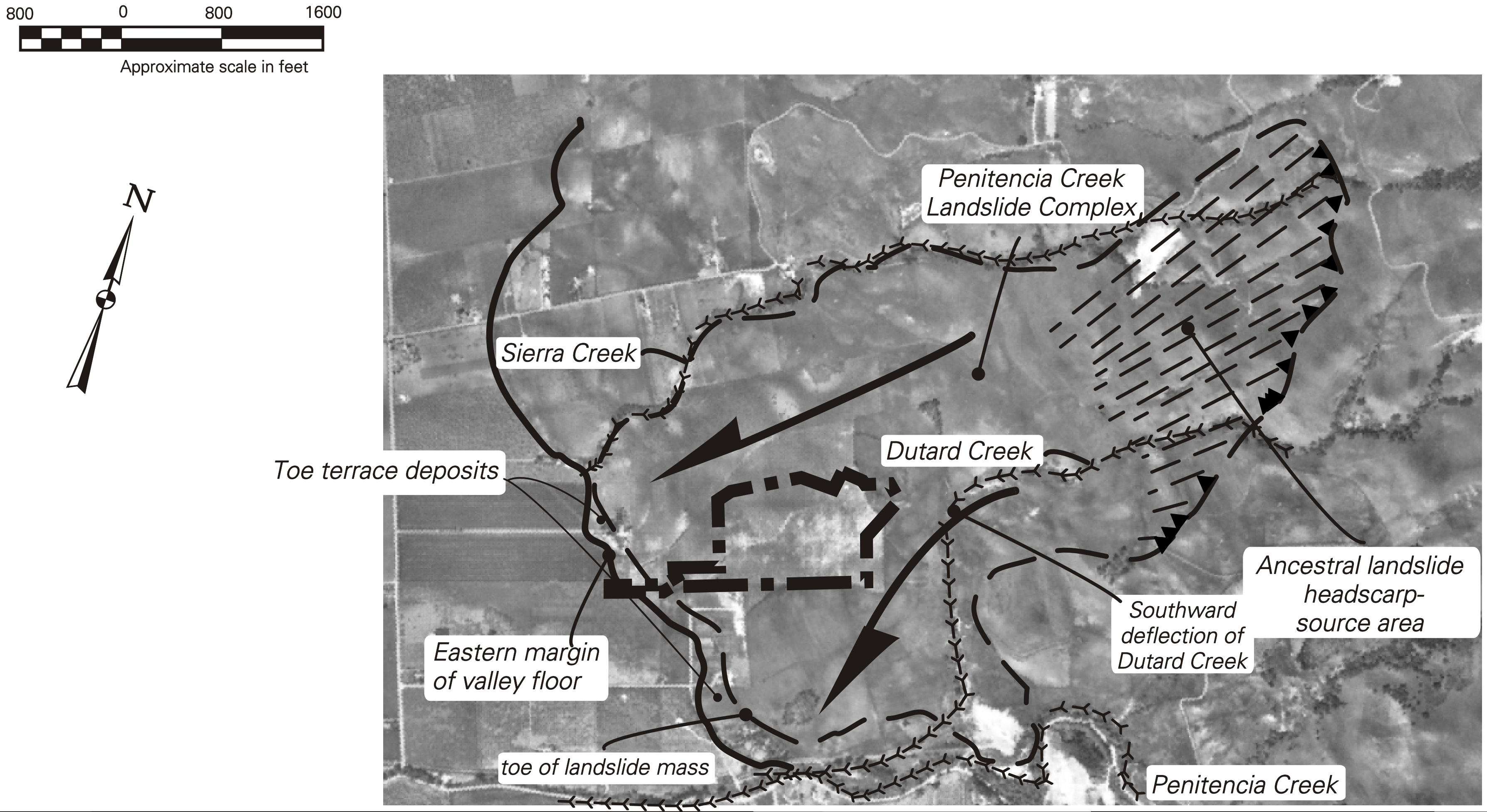
The photographs listed in Table D-1 show the Penitencia Creek landslide and vicinity under varying conditions of agricultural development, soil moisture, and cultural development. We have included 4 representative historical aerial photographs in this appendix as Figures D-1, D-2, D-3, and D-4, taken in 1950, 1954, 1963, and 1984, respectively. The earliest photographs (1950 and 1954) were chosen because they exhibit fairly good contrast and show the prominent lobate-shape of the landslide toe. These photographs also show the site prior to development. The later photographs (1963 and 1984) show the property at various stages of development. On all four photographs we show pertinent mapped features of the landslide based on our interpretation.

Table D-1
Aerial Photographs

Photo No	Date	Responsible Agency	Photo Type	Roll & Frame	Scale
1	07/31/39	U.S.D.A.	b & w	CIV-284 24, 25, 26, 27, & 28	1:20,000
2	07/31/39	U.S.D.A.	b & w	CIV-284 53, 54, 55, 56, 57, & 58	1:20,000
3	06/12/40	U.S.D.A.	b & w	CIV-346 49, 50, 51, & 60	1:20,000
4	09/26/48	U.S.G.S.	b & w	GS-HR 2-96, 97, 98, 99, 100, 101, & 102	1:23,600
5	09/26/48	U.S.G.S.	b & w	GS-HR 2-152, 153, 154, 155, 156, & 157	1:23,600
6	04/01/50	U.S.D.A.	b & w	CIV-16G 136, & 137	1:20,000
7	04/01/50	U.S.D.A.	b & w	CIV-17G 10, 11, & 12	1:20,000
8	08/07/53	U.S.G.S.	b & w	GS-YF 3-76, 77, 78, 79, 80, & 81	1:23,600
9	03/02/54	Pacific Surveys Aerial	b & w	AV- 129 13-11, 12, 13, & 14	1:~9000
10	03/02/54	Pacific Aerial Surveys	b & w	AV- 129 14-12, 13, 14, & 15	1:~9000
11	06/12/56	U.S.D.A.	b & w	CIV-6R 148, 149, & 150	1:20,000
12	06/12/56	U.S.D.A.	b & w	CIV-6R 171, 172, & 173	1:20,000
13	03/07/58	Pacific Aerial Surveys	b & w	SF Area 3-167, 168, & 169	1:35,000
14	08/15/60	U.S.G.S.	b & w	GS-VACY 2-71, 72, 73, 74, 75, & 76	1:30,000
15	07/23/63	Pacific Aerial Surveys	b & w	AV 550-16-36 & 37	1:~40,000

Table D-1 (cont.)
Aerial Photographs

Photo No	Date	Responsible Agency	Photo Type	Roll & Frame	Scale
16	09/28/63	U.S.D.A.	b & w	CIV-6DD 120 & 121	1:20,000
17	06/14/68	U.S.G.S.	b & w	GS-VBZK 3-39, 40 & 41	1:30,000
18	07/17/68	U.S. Air Force	b & w	059V 068, 069, & 070	1:~100,000
19	03/14/69	unknown	b & w	PH1065 511-18, 19, & 20	1:6,000
20	10/12/71	Pacific Aerial Surveys	b & w	AV 1006-16-08, 09, 10, & 11	1:~13,000
21	10/14/74	U.S.G.S.	color	13-107 & 108	1:20,000
22	04/12/80	U.S.D.A.	b & w	06085 179-145 & 146	1:40,000
23	07/23/80	Pacific Aerial Surveys	b & w	AV 1905-17-02, 03, & 04	1:12,000
24	02/22/81	U.S.G.S.	b & w	GSVEZR-3-32, 33, & 34	1:~26,600
25	02/22/81	U.S.G.S.	b & w	GSVEZR-3-73, 74, & 75	1:~26,600
26	07/05/82	NASA	color-infrared	HAP 82 F 237-91, 92, & 93	1:60,000
27	03/06/83	unknown	b & w	832186 1-6 & 7	1:7,200
28	08/28/85	unknown	color	S.C.V.W.D. 91765 1-1, 2, & 3	1:50,000



Date of photography: 04/01/50

File: W:\geology\penitencia creek landslide\figures\figD1.cdr

DATE	12/01/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

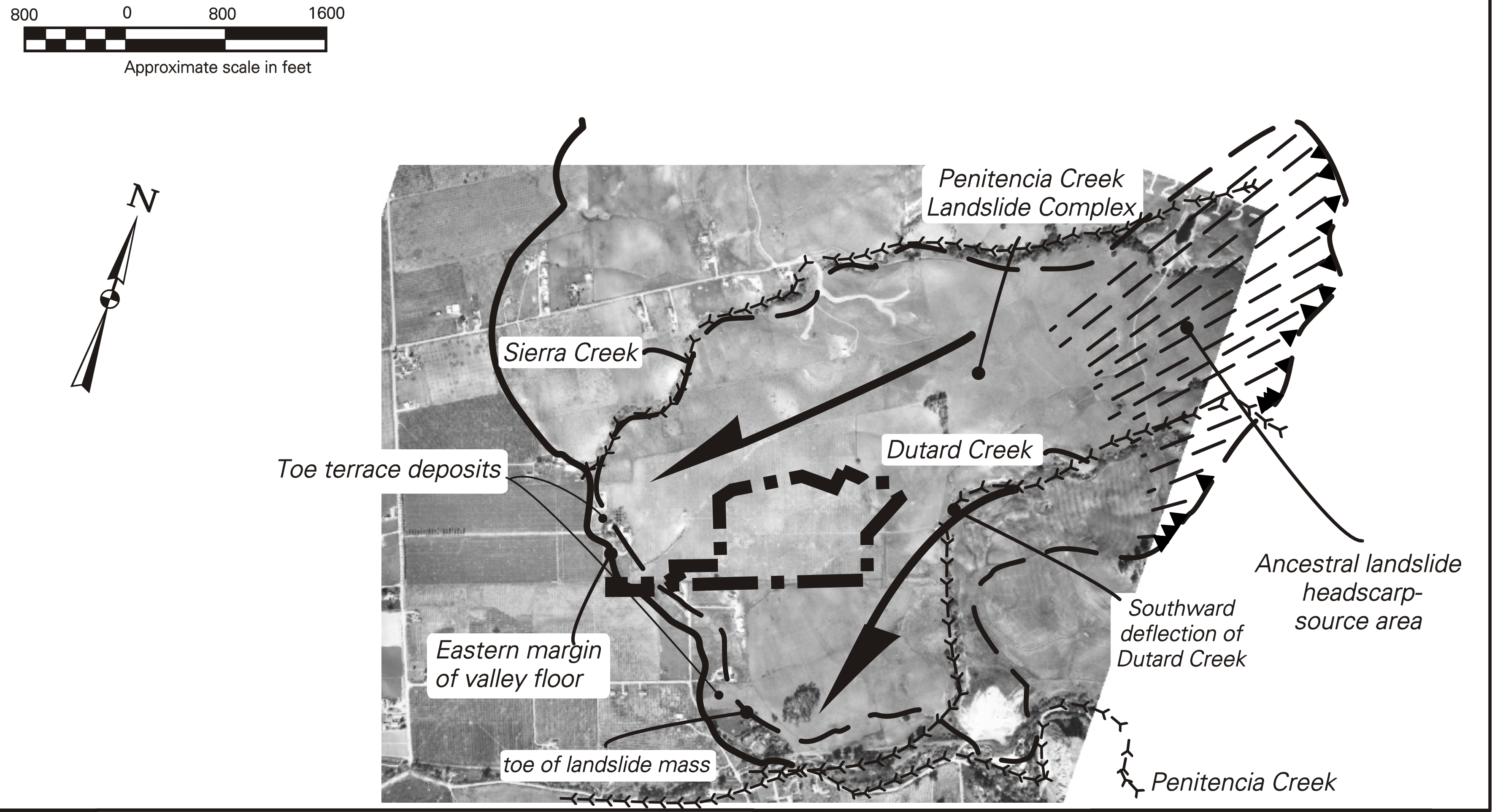
Santa Clara Valley Water District

1950 Aerial Photo Geologic Map

Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=800'

FIGURE
D-1


800
0
800
1600
Approximate scale in feet
N
Penitencia Creek Landslide Complex
Sierra Creek
Dutard Creek
Toe terrace deposits
Ancestral landslide headscarp-source area
Southward deflection of Dutard Creek
Eastern margin of valley floor
toe of landslide mass
Penitencia Creek

Date of photography: 03/02/54

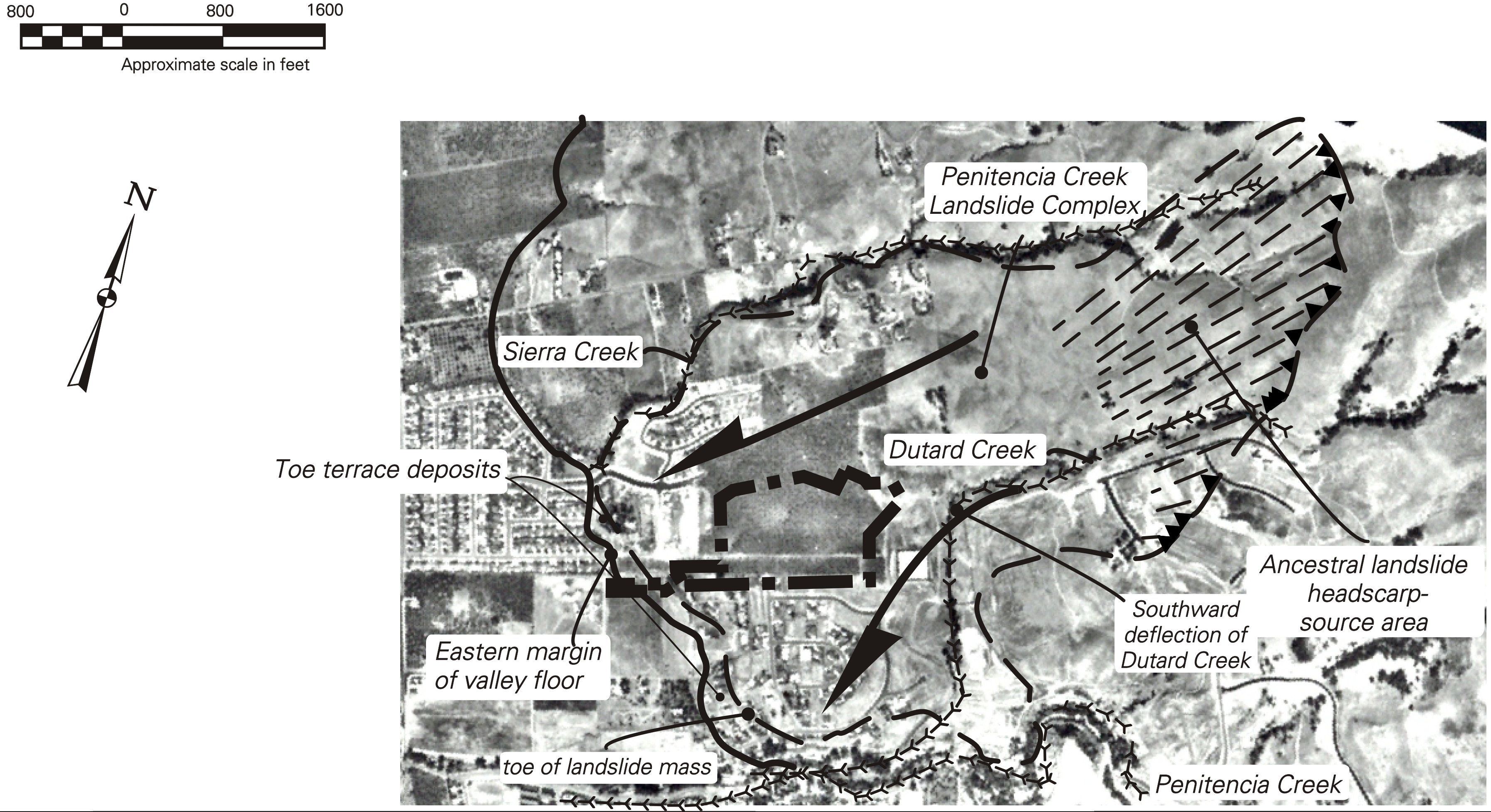
File: W:\geology\penitencia creek landslide\figures\figD2.cdr

DATE 12/01/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

1954 Aerial Photo Geologic Map
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=800'
FIGURE D-2


800
0
800
1600
Approximate scale in feet
N
Penitencia Creek Landslide Complex
Sierra Creek
Dutard Creek
Toe terrace deposits
Eastern margin of valley floor
toe of landslide mass
Southward deflection of Dutard Creek
Ancestral landslide headscarp-source area
Penitencia Creek

Date of photography: 9/28/63

File: W:\geology\penitencia creek landslide\figures\figD3.cdr

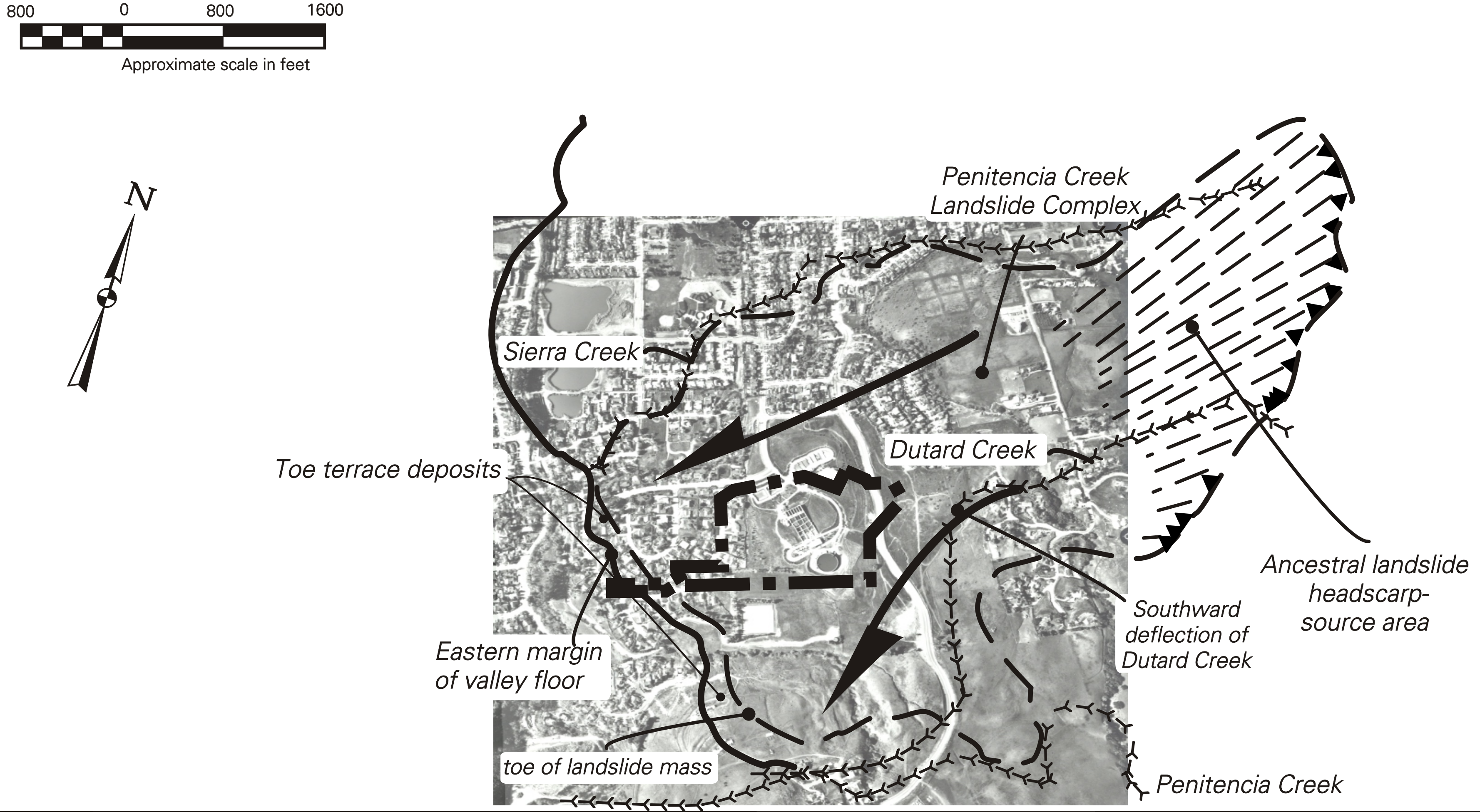
DATE	12/01/00
DESIGN	JLN
DRAWN	JLN
CHECKED	RLV

Santa Clara Valley Water District

1963 Aerial Photo Geologic Map
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE
1"=800'

FIGURE
D-3



Date of photography: 12/08/84

File: W:\geology\penitencia creek landslide\figures\figD4.cdr

DATE 12/01/00
DESIGN JLN
DRAWN JLN
CHECKED RLV

Santa Clara Valley Water District

1984 Aerial Photo Geologic Map
Penitencia Creek Landslide
Penitencia Water Treatment Plant

SCALE 1"=800'
FIGURE D-4